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PURPOSE OF THE INSTITUTE.

It is the purpose of the Institute, by means of conventions and publications, to increase and disseminate knowledge in regard to the use of cement, concrete and allied products; to conduct research into their properties and uses; and to formulate recommended practice and standard specifications.

Its objects are purely educational.

* In place of B. F. Affleck, resigned.
† In place of W. L. Church resigned.

JOURNAL
OF
AMERICAN CONCRETE INSTITUTE

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NATIONAL ASSOCIATION OF CEMENT USERS
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The Institute is not responsible, as a body, for statements and opinions advanced in its publications

VOL. II | DECEMBER, 1914 | NO. 7

ELEVENTH ANNUAL CONVENTION

The Eleventh Annual Convention of the Institute will be held at the Auditorium Hotel, Chicago, Ill., February 9–12, 1915. This Convention will mark the completion of the tenth year of the existence of the Institute, and an especially interesting and profitable program is being arranged. The sessions will be at 10 A. M., 3 P. M. and 8 P. M.

TENTATIVE PROGRAM—ELEVENTH ANNUAL CONVENTION

Tuesday, February 9, 1915. 10.00 o'clock, A. M.

Concrete Roads and Bridges.

Opening Address of Welcome.

Review of Present Practice in Concrete Road Construction.

Percy H. Wilson, Secretary, Association of American Portland Cement Manufacturers, Philadelphia, Pa.

The Service Test Concrete Road in Philadelphia.

William H. Connell, Chief, Bureau of Highways and Street Cleaning, Philadelphia, Pa.

Reinforcing Narrow Concrete Roads.

John W. Mueller, Civil Engineer, New Castle, Ind.

Report of Committee on Concrete Roads.

A. N. Johnson, Chairman.

3.00 o'clock P. M.

Concrete Roads and Bridges.

Organization and Methods of Constructing Concrete Roads.
William Acheson, Division Engineer, New York State Highway Commission, Buffalo, N. Y.

Concrete Roads and Frost Action.
A. M. Lovis, First Assistant Engineer, Massachusetts Highway Commission, Boston, Mass.

Cost of Construction and Maintenance of Concrete Roads.
H. J. Kuelling, County Highway Commissioner, Milwaukee, Wis.

The Construction of Integral Curbs.
Charles E. Russell, City Engineer, Highland Park, Ill.

Comparative Cost of Concrete Roads.
Percy H. Wilson, Secretary, Association of American Portland Cement Manufacturers, Philadelphia, Pa.

8.00 o'clock P. M.

Test of a Reinforced Concrete Slab Bridge.
E. B. McCormick, Mechanical Engineer, Office of Public Roads, Washington, D. C.

Standard Designs for Concrete Highway Bridges and Culverts.
C. B. McCullough, Designing Engineer, Iowa State Highway Commission, Ames, Ia.

Report of Committee on Standard Specifications for Concrete Highway Bridges and Culverts.
Willis Whited, Chairman.

Wednesday, February 10, 1915. 10.00 o'clcok, A. M.

Concrete and Reinforced Concrete Tests and Design.

Report of Committee on Concrete and Reinforced Concrete.
A. E. Lindau, Chairman.

Report of Committee on Nomenclature.
F. C. Wight, Chairman.

Report of Committee on Insurance.
J. P. H. Perry, Chairman.

The Design of Wall Columns and Wall Beams.
Edward Smulski, Consulting Engineer, Boston, Mass.

12.00 o'clock Noon.

Close of Ballot Box.

3.00 o'clock P. M.

Reinforced Concrete Stands, Stadias and Bowls.
Charles W. Leavitt, Civil and Landscape Engineer, New York, N. Y.

Design and Construction of the Massachusetts Institute of Technology Buildings.
Sanford E. Thompson, Consulting Engineer, Newton Highlands, Mass.

The Design of Concrete Bins for the Storage of Cement.
Leonard Wesson, Assistant to President, Universal Portland Cement Company, Chicago, Ill.

The Manufacture and Laying of Concrete Sewer Pipe.
Henry T. Shelley, Assistant Engineer in Charge of Sewer Construction, Philadelphia, Pa.

Tests on Egg-Shape and Circular Reinforced Concrete Sewer Pipe.
Albert T. Goldbeck, Engineer of Tests, Bureau of Surveys, Philadelphia, Pa.

8.00 o'clock P. M.

Concrete and Reinforced Concrete Tests and Design.

Annual Address by the President.

A Critical Review of Current Practice in Reinforced Concrete as Embodied in Building Regulations and the Joint Committee Report.

Edward Godfrey, Structural Engineer, Robert W. Hunt and Company, Pittsburgh, Pa.

Basis of Current Practice in Design of Reinforced Concrete Structures.

C. A. P. Turner, Consulting Engineer, Minneapolis, Minn.

A New-Old Theory for Reinforced Concrete in Bending.

L. J. Mensch, Contracting Engineer, Chicago, Ill.

Thursday, February 11, 1915. 9.00 o'clock A. M.

Business Session.

Annual Report of Board of Direction. Report of Tellers.

10.00 o'clock A. M.

Concrete in Art and Architecture.

Report of Committee on Cement Products.

C. K. Arp, Chairman.

Cement Roofing Tile.

J. S. Freund, General Manager, American Cement Tile Manufacturing Company, Pittsburgh, Pa.

Some Principles of Design as Applied to Concrete Houses.

Charles E. White, Jr., Architect, Chicago,

The Fireproof Home.

R. P. Stoddard, Executive Secretary, Society Advocating Fire Elimination, Cleveland, Ohio.

3.00 o'clock P. M.

Report of Committee on Treatment of Concrete Surfaces.
L. C. Wason, Chairman.

A New and Simple Method for Securing Dustless Concrete Floors.
P. M. Bruner, P. M. Bruner Granotoid Company, St. Louis, Mo.

Topical Discussion on the Dusting of Concrete Floors.

8.00 o'clock P. M.

Report of Committee on Edison Fire.
Cass Gilbert, Chairman.

Concrete: A Medium of Æsthetic Expression.
Irving K. Pond, Architect, Chicago, Ill.

Synthetic Stone in Catskill Aqueduct Buildings.
H. Lincoln Rogers, Architect, Board of Water Supply, New York, N. Y.

Friday, February 12, 1915. 10.00 o'clock A. M.

Plant Management and Cost.

Contractor's Equipment, Austin-Nichols Warehouse.
T. A. Smith, General Superintendent, Turner Construction Company, New York, N. Y.

Mechanical Plant for Handling Concrete.
William P. Anderson, President, Ferro Concrete Construction Company, Cincinnati, Ohio.

The Strength of Concrete Forms.
Harrison S. Taft, Contracting Engineer, Seattle, Wash.

Concrete Forms for the Catskill Aqueduct.
Alfred D. Flinn, Deputy Chief Engineer, New York Board of Water Supply, New York, N. Y.

3.00 o'clock P. M.

Report of Committee on Methods of Tests and Standard Specifications for Concrete Materials.
Sanford E. Thompson, Chairman.

Mixing, Curing and Placing Concrete with High Pressure Steam.
Harold P. Brown, New York, N. Y.

Some Further Results Obtained in Investigating the Properties of Portland Cement having a High Percentage of Magnesia.
P. H. Bates, Chemist, Bureau of Standards, Pittsburgh, Pa.

8.00 o'clock, P. M.

Annual Banquet in celebration of Tenth Anniversary.

The Members are requested to distribute, where they will do the most good, the 5 copies of Tentative Program herewith enclosed.

Proposed Amendment to By-Laws.

The following proposed amendment to the By-Laws will be considered at the Business Session, at 9 o'clock A. M., on Thursday, February 11, 1915.

Change Article IX, Section 2, now reading—

"SECTION 2.—The annual dues of each member shall be five dollars."

to read—

"SEC. 2.—The annual dues of each member shall be ten dollars."

Nominations for Officers for 1915.

The Committee on Nomination of Officers submits the following nominations:

President, LEONARD C. WASON.
(To serve 1 year.)

Vice-President, WILLIAM K. HATT.
(To serve 2 years.)

Vice-President, HENRY C. TURNER.
(To serve 1 year, filling unexpired term of L. C. Wason.)

Treasurer, ROBERT W. LESLEY.
(To serve 1 year.)

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(To serve 2 years.)
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Fifth District: WILLIAM P. ANDERSON.
(To serve 1 year, filling unexpired term of Robert W. Lesley.)
Third District: ERNEST ASHTON.

FRANK C. WIGHT, *Chairman,*
B. F. AFFLECK
B. S. PEASE
CHARLES D. WATSON
RUDOLPH J. WIG.

Letter Ballot for Officers.

Enclosed will be found the Official Ballot for Officers of the Institute for the ensuing year.

The ballot must be placed in the special envelope, endorsed on back as indicated, and mailed so as to reach Chicago not later than 12 o'clock noon on Wednesday, February 10, 1915, at which time the ballot-box will be closed.

To be presented at Chicago Convention—1915.
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A CRITICAL REVIEW OF CURRENT PRACTICE IN REINFORCED CONCRETE DESIGN AS EMBODIED IN BUILDING REGULATIONS AND THE JOINT COMMITTEE REPORT.

By Edward Godfrey,*

Mark Twain once said that he had nothing against the German language, he would just like to reform it. I have nothing against reinforced concrete, I would just like to reform it.

At the outset I wish to make two statements that no sane engineer can contradict. The first is: There are entirely too many failures in reinforced concrete construction. The second is: There has been entirely too little attention paid to these failures.

Another statement which I wish to make with greatest emphasis and without reservation is this: Practically all of the reinforced concrete failures have been due to faulty design, and generally the design is standard design, accepted by the profession.

When your worthy President, Mr. Humphrey, called me up a few days before Christmas and asked me to write a paper on this subject, I am sure he was well aware of the kind of paper that he could expect. Hence I have no apology to offer for the nature of this paper, neither have I for the use of the pronoun of the first person singular—it is more direct and it saves time.

In 1910 I wrote a paper that was read before the American Society of Civil Engineers. It contained sixteen indictments against common practice in reinforced concrete design. That paper was not an expression of opinion or opinions, but a statement of the principles of proper design in reinforced concrete as demonstrated by tests and as dictated by common sense. This paper will likewise be void of opinions.

Every reinforced concrete wreck before or since that paper was read has violated one or more of the principles of good design laid down in the paper.

I have been held up as the arch enemy of reinforced concrete because I have unequivocally held that all of the big wrecks are due to bad design. Men who believe they are friends of reinforced

*Structural Engineer, Robert W. Hunt and Company, Pittsburgh, Pa.

concrete have generally strenuously held that the designs of the wrecks were without flaw. Has it never occurred to you, gentlemen, that the man who after investigating a complete failure of a reinforced concrete structure pronounces the design faultless has said the very worst thing that he could say against reinforced concrete as a form of construction?

The most discouraging thing possible to say concerning reinforced concrete, in the light of the numerous wrecks of buildings and bridges, is that there is no room for improvement in the matter of design. To say that a little sawdust in a column, or a block of wood, or a batch of bad concrete, or any such trifling imperfection in the execution of the work, is responsible for the entire collapse of a building is to put reinforced concrete in a perilous class where nothing but absolute perfection of execution will assure unfailing safety. If this were true reinforced concrete would be as safe a thing to handle as nitroglycerine. But it is not true. Reinforced concrete is capable of safe design, design that has a factor of safety to cover imperfections.

In a discussion of structural failures, a reinforced concrete engineer was given the subject of failures in reinforced concrete. He talked for some time and the substance of his talk was that reinforced concrete has not failed. He made no reference whatever to the numerous failures of buildings nor their cause. Wilful and deliberate blindness to big facts never accomplished anything. No unprejudiced engineer makes the claim or the insinuation that reinforced concrete has failed, but this is no reason why the building failures that occur in such regularity, and their lessons, should be totally ignored.

There has been some sporadic reform in reinforced concrete design, but it has been in the practice of some engineers *and it has not been in building codes or regulations.* There are some signs that engineers are waking up to the absurdity of standard design, and this gives hope that needed reforms will eventually work their way into building codes, and the Joint Committee Report and even into standard books.

Some months ago I sent a little design to an engineering paper, which was published anonymously. It was designed by the Joint Committee Report. Four engineers at once criticized the design severely. Two of them were members of the Joint Com-

mittee. The only approach to a defense of the Report was a hysterical reference to the fact that that Report defines mass concrete. It is needless to say that the design was a fake, purposely made as bad as the Joint Committee Report would allow, which was very bad. The thing would no doubt collapse under its supposed safe load. And yet (when a slight correction was made) no engineer, either of the thirty composing the Joint Committee or any other, could point out where the Joint Committee Report was violated in the least degree. Such things could not possibly be true if that Report were a safe standard of design.

I criticized the Joint Committee Report when it was first published in 1909 and pointed out the dangers of it. The only answer made by the Committee has been a second report as bad as the first.

I have tried more than once to have building code writers forbid some of the abominable things that have become standard, but the answer is a code that allows the worst of them. I have been preaching against these absurdities since 1906 in articles and letters and in my book "Concrete," but I have not as yet had many more converts than Noah after his protracted efforts. Editors tell me that I arouse antagonism. That's just what I want to do. In heaven's name, gentlemen, what would you have me do? What would you do if you saw buildings collapsing and men's lives being taken by reason of designer's stupidity, and if the cause of these wrecks were as clear as day? I would consider myself guilty of manslaughter, if, seeing the cause of these wrecks, I held my peace. One of the purposes of writing this paper is to arouse criticism. I want to get engineers to talking or writing. I have tried for years to get some men just to express themselves on certain details of design. One authority said he would like to discuss stirrups with me. I said all right, and gave him a subject and asked him in what engineering paper he would like to carry on the discussion. He made it plain months later that he merely wanted to enter into a verbal confab on the subject.

The Joint Committee Report is the father of most of the building codes and regulations and is usually considered the most authoritative word on the subject of reinforced concrete. Hence that report will be the main subject for criticism in this paper. The criticisms will be adverse, not because there is nothing to

commend in the Report, but because the faults are so glaring and have bred so many wrecks that the Report ought to be revised at once.

I have taken up a lot of space in this paper with what might be considered preliminary and historical matter and have not yet singled out any particular fault in building regulations and the Joint Committee Report. It may be thought that this time could be better employed in demonstrating or attempting to demonstrate the errors of modern reinforced concrete design. But the matter is a psychological one—wilful blindness is psychological. It is beyond the influence of mathematical demonstration or of common sense. The demonstrations are easy, absurdly so. Engineers have practically ceased to combat them. The reform is due. It is only a matter of when or how. Will the engineering profession reform itself? or will external pressure be necessary to effect this reform? Will the general public be so roused by the oft recurring failures that they will demand that an end be put to it by adopting safe design in building codes?

The Joint Committee Report allows plain concrete columns. Columns—just think of it!—shafts twelve times as high as their width; and it allows them to be loaded to about 500 lb. per sq. in. Just imagine a building with posts 10 in. sq. and 10 ft. high each of which is loaded with 50,000 lb.! How many engineers present would be willing to sleep in such a trap? Yet this is the sober manifesto of 30 prominent engineers. It is not a matter of opinion that such columns in a building would be outrageously bad. It is simply common horse sense. It would be a waste of time to attempt to prove that such columns are utterly unfit to support a building. And yet it would be possible, with infinite care, to put up a building with columns of this sort, and it might stand up and carry its load.

The Joint Committee Report allows the cousin to the plain concrete column, the rodded column. This is a plain concrete column with a little rod in each corner, the rods being held in place "during the pouring of the concrete," with some little wires. These wires could be of broom-wire size and spaced several feet apart and the Report would be completely satisfied. By the plain wording of the Report these wires have no structural office in the column. They are merely to hold the upright rods in place during

the pouring of the concrete. They can therefore be ignored in a discussion of this column.

Is it a matter of opinion that this column is bad? There are several ways of going about to demonstrate the fitness or unfitness of a structural member to perform its office. First there is analysis. From the viewpoint of analysis what has the rodded column to commend it? Does a long slender rod strengthen the concrete or hold it together? Not one iota. The concrete can break out in chunks, or shear diagonally with just as much ease with this rod present as not. In fact the rod under compression helps the concrete to break out—actually breaks it out. This is what analysis would anticipate; it is what practice demonstrates. If the rod does not hold the concrete nor aid it in any way, it follows that the concrete must aid the rod. Is this reinforced concrete or reinforced steel? Is a long slender rod in compression held in line by a little concrete any kind of structural member that is deserving of serious consideration?

The tendency in a concrete shaft in compresson is to bulge out and spall off in chunks. An upright rod does not even tend to prevent this, hence it does not reinforce the concrete by overcoming its weakness. From every standpoint analysis would put the rodded column clear outside the pale of safe construction. What do tests have to say about it?

Only a few tests will be cited. So far as I know there are no recent tests to cite. Investigators are getting wise to the fact that *the rodded column is no longer seriously defended by any engineers who value their reputation.* The error of the rodded column is beginning to be appreciated. The most recent investigation into the strength of reinforced concrete columns deliberately ignored the rodded column.

In Bulletin No. 11 of the University of Illinois Engineering Experiment Station there are some tests on plain and rodded concrete columns. In a number of cases the concrete column with no steel whatever stood more load by a large percentage than rodded columns of the same size and the same pouring of concrete. This is not opinion; it is cold fact. And yet that same bulletin uttered some opinions as to how much these concrete columns were reinforced by the rods that brought on their premature destruction.

What does experience tell us of the rodded column? This is the blackest page of reinforced concrete history. With scarcely any exceptions the big wrecks have been of rodded column installations and the columns invariably demonstrate their unfitness by not only failing to stand up and carry their load but by breaking up into chunks with their so-called reinforcing rods curled up in such way as to demonstrate clearly their futility. It is scarcely necessary to go into the details of these wrecks. Every engineer who reads is familiar with the appearance of these so-called reinforced concrete columns after something has touched off a structure in which they are the supporting members and after that structure has collapsed into a mass of ruins. If they were real columns and not fragile shafts of artificial stone, they would at least stand up. They are not even capable of standing upright nor of remaining in one piece.

How have rodded columns stood in a fire? Look at the Edison fire. A large portion of one of the buildings collapsed. A larger portion stood up after the fire as by a miracle. The columns were split from top to bottom; the reinforcing rods were bowed out and entirely free from the columns. Reinforced concrete did not prove a failure in that fire, but the rodded column did prove an ignominious failure.

The rodded column is wrong from every point of view. Increasing the size or reducing the unit stress does not remedy it. Big clumsy columns break up in a wreck just as the slender props do. In one building a seasoned column 75 ft. away from the wrecked portion broke under 200 lb. per sq. in. merely from the shock of the wreck.

It is wrong to allow any compression whatever on slender rods in concrete *and standard works that recommend that from* 15 *to* 39 *per cent additional load for every one per cent of steel be allowed are simply committing egregious errors.*

The rodded columns of the wrecks have in many cases been those in which there are wires or small rods spaced about a foot apart. The behavior of the columns and the tangled and twisted shape of the supposed reinforcing rods are proof enough that this sort of column is totally lacking in the essentials of a good column.

It is true that if extraordinary care be taken, it is possible

to construct a building having rodded columns that will stand up and carry its load. But to say that it is therefore a safe column is equivalent to saying that it is safe to commit murder because so many murderers go free.

The safe way to design a reinforced concrete column is to use a spiral or hoops and upright rods. Not "rectangular" hoops as one proposed building code had it. To call a wire bent in a square or oblong shape a hoop and a column having these wires a hooped column is a libel on both common sense and the hooped column. Imagine a barrel with square hoops. A column with so-called square hoops is just as rational.

Building regulations and the Joint Committee Report do not correctly state the compressive strength of the hooped column, and the unit stresses used are higher than the facts warrant. It is true that a hooped column will exhibit a large ultimate strength before it finally crushes, but the point of incipient failure of a hooped concrete column is identical with the point of failure of plain concrete. I pointed this out as being demonstrated by tests in *Proc. Am. Soc. C. E.*, April, 1914, p. 1092. The point of incipient failure (that is, where the column begins to crack) is the point of ultimate usefulness of a column. Hence this should be considered the ultimate strength and the factor of safety based thereon.

Considère's formula for the strength of a column, as given in the building regulations, is wrong. That formula gives three elements in the strength of a column: first, the concrete of the shaft; second, the upright rods; third, the hoops. Now, the concrete and the reinforcement are in compression and by altering these the strength of the column will be altered. But if the hoops are of a given pitch, it does not affect the strength of the column one iota to add to the section of the hoops, assuming that the original hoops are not deficient. It is a matter that needs no demonstration that the strength of the disc between two hoops is totally independent of the sectional area of the hoops and that the entire compression of the column passes through this disc.

The proper way to make requirements for a hooped column is to establish a standard size and pitch of hoops for a given diameter of column and then allow a certain unit stress on the concrete. The upright rods should also be standard. It would

be easy for a designer to use a light wire with a wide pitch for his hooping or coil and claim the advantage of the hooped column in unit stress when his column might in fact be as bad as the rodded column. The upright rods and the coil are merely a cage to confine the concrete and give the column needed toughness so that simple compression exists in the concrete and it is rendered free from shearing and bulging strains. A column with this cage around it meets the definition of reinforced concrete as given in the first sentence of your Standard Building Regulations, which says that reinforced concrete is an approved concrete which has been reinforced by metal in some form so as to develop the compressive strength of the concrete.

The rodded column does not meet that definition by any strength of imagination, and the sooner this is made manifest the better. I tell you, gentlemen, the days of the rodded column are numbered, and if any of you do not want to be guilty of manslaughter, you will drop it immediately. The Edison fire and the Youngstown Hippodrome failure are enough in themselves to condemn forever this absurd blunder of the engineering profession. Read the lefthanded recognition that the rodded column got last year in the Western Society of Engineers. Read editorial comments on the Edison fire and the Hippodrome failure. Engineering editors generally wait until the profession is ready to receive an innovation before they spring it. How is this comment concerning the failure named above? "In the matter of beam reinforcement and connections and column reinforcement the design follows standard practice and is at fault only so far as that practice is at fault."

I am here, gentlemen, to tell you as I have been telling the profession for eight years, that the standard practice is woefully at fault in all of these matters. I shall touch on the beam reinforcement shortly.

The hooped column, besides being a true reinforced concrete member, has a factor of safety against a fire which is characteristic of all true reinforced concrete. A fire would have to burn a long time before the outer shell of concrete over the hoops would be destroyed. Water on the hot shell might cause it to break off, but still the rods would be there confining the inner concrete core, and the carrying capacity of the column would be great.

Another common error in design is dependence upon stirrups or shear members, and another fault in building regulations and the Joint Committee Report is the recognition of the stirrup or shear member. The stirrups have been given such fanciful offices as abutments to imaginary arches, vertical or diagonal members in imaginary trusses, etc., but there does not exist in engineering literature any approach to a logical analysis showing how a stirrup or a shear member can take the stress attributed to it.

The stirrup is on a par with the rodded column, and dependence on stirrups to take the shear of a beam is just as indefensible as dependence on rodded columns. Furthermore, beams that have been designed with no better shear reinforcement than the stirrup or shear member have failed ignominiously in wrecks and have been the cause of some of them. The shear member is another of the things that engineers have ceased to defend, though I have been emphasizing its absurdity for eight years. Yet these same engineers and experts will go on using the shear member, and when a wreck occurs that shows the absolute foolishness of dependence upon shear members, they will declare that the design of the fallen structure was perfect.

Paragraph 74 of your Standard Building Regulations provides that "members of web reinforcement shall be embedded in the compression portion of the beam so that adequate bond strength is provided to fully develop the assumed strength of all shear reinforcement." This sounds plausible until it is analyzed. If it were practicable to anchor the shear member in the small fraction of the depth of a beam above the assumed neutral axis, which it seldom is, what would then become of the shear? Would it not have to be taken to the support by the concrete after all? What then is the use of the shear member? This same paragraph says that "web reinforcement, unless rigidly attached, shall be placed at right angles to the axis of the beam and looped around the extreme tension member." I see but one answer to this: the "extreme tension member" must carry the shear up to the vertical shear member. This is quite untenable.

From the standpoint of analysis the shear member has no standing whatever.

From the standpoint of tests the shear member has failed also. Tests have proven that shear members have no office,

practically no stress, until the beam has failed. Even during failure the shear member in the very critical section of the beam is useless, because it is located in such way as not to cross that section and have anchorage on each side of it.

The Joint Committee Report does not state whether the stress in a shear member is shear or compression or tension. All three are equally absurd. Your regulations indicate that it is tension. Some books still give it as shear; most of them vaguely give it as "stress." What do tests on a finished building find as

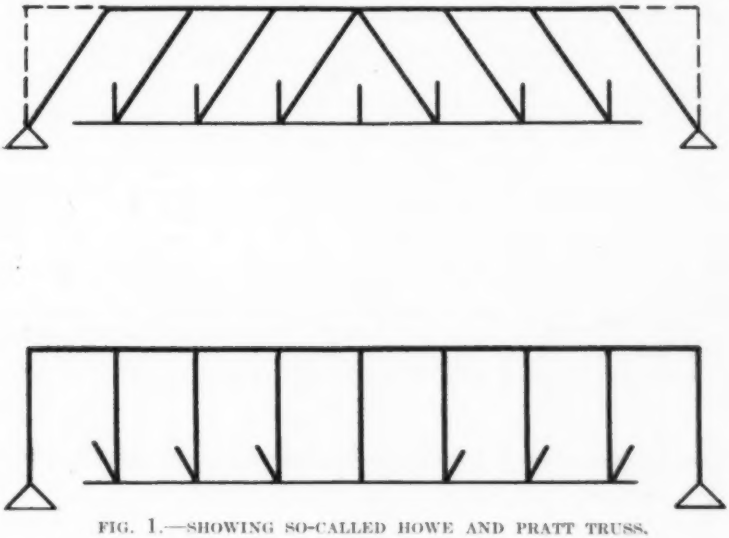


FIG. 1.—SHOWING SO-CALLED HOWE AND PRATT TRUSS.

the stress in stirrups? Compression! As the lawyer examining an expert witness would say, That will do.

Fig. 1 illustrates the so-called Howe and Pratt truss action of systems of shear members. The heavy lines represent the compression members formed by the concrete. The light lines represent the shear members and the extent of their usefulness. Is any comment needed?

Fig. 2 shows several methods of "reinforcing" a beam for shear, including bending up a main tension rod and stopping it at the support. The heavy line shows how the beam could break away from the support by merely pulling out the little ends of a

few of the shear members, leaving the others entire and utterly useless. This is the way a number of failures have taken place, notably the Youngstown Hippodrome. In that building I saw where beam after beam had broken away from wall or column leaving no projecting steel.

What is the remedy? It is simply this. Instead of using stirrups or shear members, run the main reinforcing rods up to the

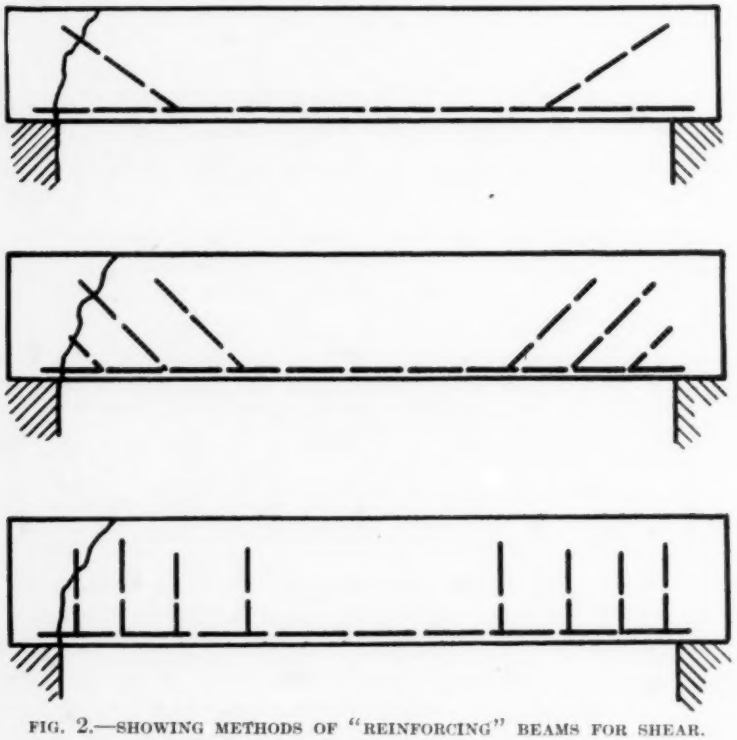


FIG. 2.—SHOWING METHODS OF "REINFORCING" BEAMS FOR SHEAR.

top of the beam and into the next span, or into the slab, or down again into the column or supporting girder for anchorage. It is not a matter of opinion but the commonest kind of horse sense that a beam so built could not fall away from its support without severing the main steel rod or pulling that rod out of a deep embedment and full anchorage in concrete.

The history of reinforced concrete would have been vastly

different if the rodded column and the stirrup or shear-member cluttered beam had never been invented.

Another folly in the Joint Committee Report is compression in slender rods in a concrete girder. The same argument holds

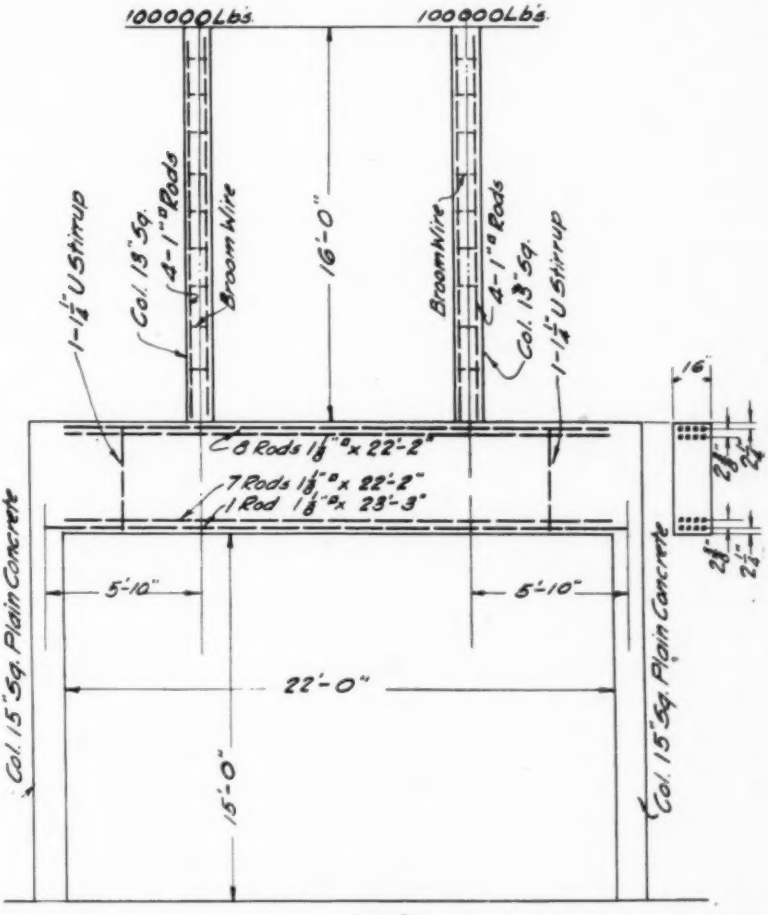


FIG. 3.

here as in the case of the rodded column. The only way these rods could be held against bulging out is by close spaced hooping, and this is out of the question. If the limitations are such that adequate compression area cannot be had in the concrete, it simply

means that the thing to use for that particular girder is steel construction. There is no special virtue in being able to say that "this building contains no structural steel whatever." The true engineer will use the thing most suitable to solve his problems, and the sooner others discard partisan attitudes the better for the public good.

Fig. 3 is the little design that I had published anonymously as the apparent attempt of a raw recruit at designing by the profession's high standard, the Joint Committee Report. Would any engineer here father it? Here we have the plain column, the rodded column, the stirrup, the rods in compression. Another fault in this design is the bunching of rods in the tension portion of the beam which the concrete is incapable of gripping. This is another gross error in the Joint Committee Report. It takes little in the way of argument or proof to demonstrate the error. If anyone has an argument that would prove that the little plastering of concrete surrounding the heavy rods of this girder can grip those rods to the extent of 144,000 lb., let him bring it out. If not, go and use your influence to have the Joint Committee Report revised.

The formula in paragraph 57 of your Regulations showing the supposed distribution of loads in a rectangular slab, while it is common, and imported, and all that, is wrong. It is not based on correct mathematical deduction and is not in agreement with tests. It should be eliminated and some formula substituted that will give the full load across the short axis when the ratio is 2 to 3.

The latest utterance of the code writers is the Chicago-Flat-Slab Ordinance, said to be the result of four years of study and test. I have shown in a letter recently published in *Engineering News* that besides needing private instruction in its interpretation this ordinance falls far short of supplying the reinforcement needed for the static stresses of a flat slab, though a flat slab partisan has since stated that a slab would be much too safe if designed by it.

In Fig. 4, the sume of the bending moments on the lines NQ and MP, or OR (considered portions of sections of indefinite extent) is by the principles of statics a definite amount, namely, the load per sq. ft. multiplied by one-eighth of the square of the span between MP and OR. This is a mathematical certainty and

not a matter of opinion or "views," as an engineer has recently claimed. Placing in the slab the reinforcement required by the Ordinance leaves it far short of the necessary reinforcement. Why should the flat-slab user have this advantage over the user of any other slab, when the only difference in his reinforcement is that some rods lie in a diagonal direction?

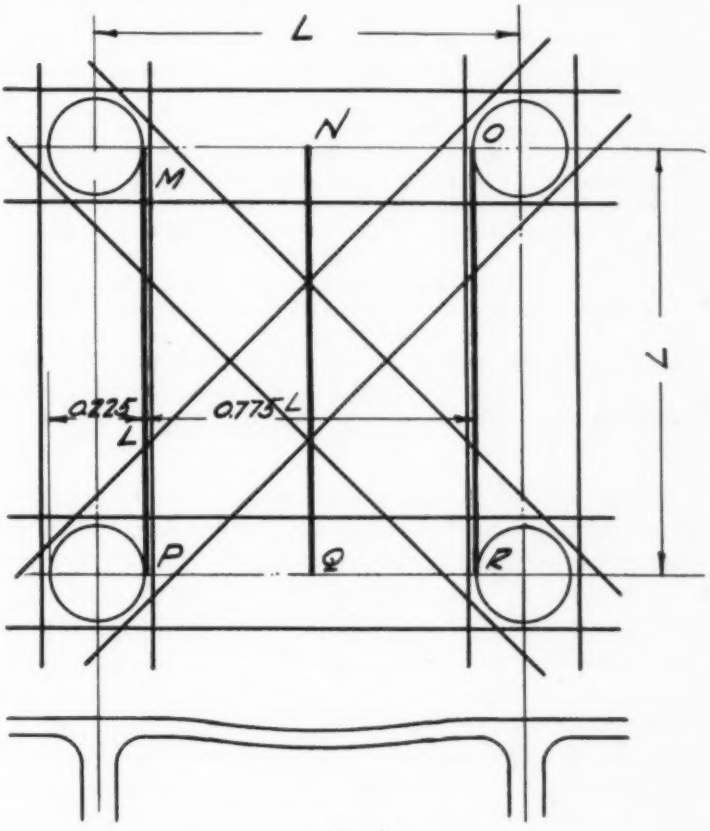


FIG. 4.

A flat slab on rows of columns is no better conditioned than the same slab on lines of girders. Building regulations are clear as to what reinforcement is needed in a slab across lines of girders; why do they require only a fraction of this for a slab across lines of posts?

I have a number of times asked the proponents of the flat slab theories why the flat slab on posts should be considered, in designing, as superior to a slab on girders, and why it would not deflect in approximate cylindrical shape, if a line of bays across a building or in a freight terminal were loaded to their alleged capacity, for it is true that no commercially designed flat slab will stand this criterion, nor will a slab designed by the Chicago Code. Mr. Eddy has made two answers, both of which are evasive. One answer is in *Trans. Am. Soc. C. E.*, Vol. LXXVII, p. 1424. He says:

"Suppose, for example, a long building with two rows of columns parallel to the long sides, and panels nearly square, so that there are three tiers of panels lengthwise of the building, one long interior tier being between the two long wall tiers. Now, suppose heavy loads to be placed on three panels of a tier or bay extending across from wall to wall. If the writer understands Mr. Godfrey, he thinks that such an arrangement of loading would produce a critical stress in a mushroom slab much greater than if the loading were removed from either two of the three panels. It might do this were it not for the wall supports and the stiff heads; but in fact the reverse is true. The walls entirely prevent the formation of the ends of any such cylindrical trough across the slab, and the stiff heads and walls will tend to accentuate the hollows parallel to the long walls, rather than those perpendicular to them."

Note that instead of being a row of mushroom panels this is only three and that two of them are exterior panels supported on walls and not on columns. Also note that by Mr. Eddy's own statement it is the walls that are relied on to save the mushroom slab from a condition of critical stress. Why did he not assume that the middle row of panels in this long building was loaded, so as to eliminate the effect of the walls? No test has ever been made under this condition.

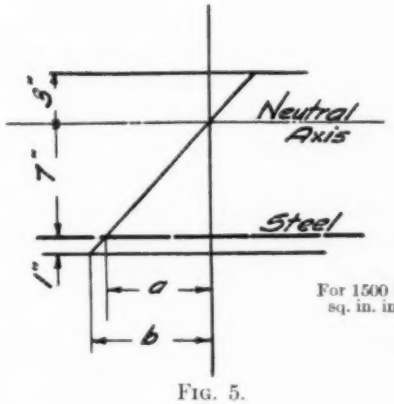
Mr. Eddy's second evasive answer is in *Trans. Am. Soc. C. E.*, Vol. LXXVII, p. 1714. I had made this proposition:

"Given two rows of columns and a flat slab over their tops, acting as a viaduct to carry a railroad track midway between them. Suppose the jarring of the moving load has cracked the slab down the middle of the track or that contraction of the concrete has produced the same result. Now apply any commercial flat-slab theory with its 'interior-bay-surrounded-by-balanced-loads' species of analysis. Next apply the theory of flexure to this slab—a simple beam supported along two lines (the two rows of columns), balanced beyond these lines with dead load only. The bending moment along the

crack is simple and perfectly definite. All the tension to resist this bending moment must be taken by the reinforcing steel. If the slab has been designed by any commercial flat-slab theory, the stress in the steel will be found to be such that failure would be looked for."

In Mr. Eddy's reply, above cited, he simply makes the panels oblong, with the long axis in the direction of the track, so that a failure would be pulled off in the slab crosswise before it could be severely tried in the other direction. This is not sincere discussion: no oblong slab was hinted at in my proposition.

To return to the Chicago flat-slab code. The code writers arrived at their conclusions at the end of their four years' study



Concrete Area (1 in. width) = 10 sq. in.
Steel = Area .05 sq. in.

a = Concrete Unit Stress 100 lb. per sq. in. for each 1500 lb. per sq. in. in Steel.

b = Concrete Unit Stress $\frac{8}{7} \times 100 = 114$ lb. per sq. in. for each 1500 lb. per sq. in. in Steel.

For 1500 lb. per sq. in. in Steel. } Steel Tension = $.05 \times 1500 = 75$ lb.
Concrete Tension = $114 \times 4 = 456$ lb.

FIG. 5.

from the results of tests, some of which were on finished buildings. These tests were largely with moderate loads. Their measured stresses in the reinforcement were generally while the concrete was still whole and taking tension. Fig. 5 shows the aid supplied by tension in the concrete in a sample slab with $\frac{1}{2}$ of one per cent of reinforcement. Six times as much tension is taken by the concrete as that taken by the reinforcement. *Only one-seventh of the tensile stress carried by the reinforcement.* Is it any wonder that in a whole slab the reinforcement stresses are light. A measured reinforcement stress of 3000 lb. per sq. in. would mean potential reinforcement stress of 21,000 lb. If the investigators concluded that five times the load could be put on the slab before the "safe"

stress of 15,000 lb. in the reinforcement were reached, this would mean a potential stress of 105,000 lb. per sq. in. which would then be actual, as the concrete could not stand the extension.

There is no mystery whatever about the great strength exhibited by some flat-slab tests and no need to bring in Poisson's ratio to explain it nor to go off in flights of mathematical yiddish. Tension in the concrete is a complete explanation of it. But there is a jumping-off place where cracks in enough directions will throw that great predominance of the tension into the reinforcement and the results will be quite natural and simple.

The Chicago Code makes no provision for taking the tremendous bending stresses in the columns that must exist according to the proportions of the flat slab. This is another gross error. The bending moments are there or else there could not be equilibrium. To ignore them is inexcusable. One defender says that the provision that columns be one-twelfth of the span in diameter makes them many times stronger than necessary. Does it? Here is what I found in a flat-slab building that I checked recently. The exterior columns were not good for one half of the direct load by the plain requirements of building code. And these columns were precisely one-twelfth of the span in diameter. Furthermore they were rodded columns.

Some weeks ago I wrote a letter, which was published in the *Engineering News*, criticizing the Chicago flat-slab ordinance. A number of defenders have been heard in response through the same medium (See *Eng. News*, Dec. 24, 1914), but my reply to their comments on my letter was returned to me. It is in place to incorporate the points of that reply in this paper, as the subject matter is in line with the title of the paper. Some replies have already been given.

One sponsor for the Code disposed of the matter of bending moments in columns by a sketch showing the supposed manner in which bending moments are scattered among the columns of a building and even the column footings. The idea seems to be that because there are so many columns in a building that the bending moments would be lost or dissipated. And yet this sketch shows points of inflection in the slab one-quarter of the span from the column. This means, with special reference to the exterior columns, that a large part of their load is applied one-

quarter of the span away from the column. Now if the direct load of a column is applied only one-sixth of the column diameter off center, the unit stress is doubled. If it is applied one-quarter of the span away, when the span is twelve times the diameter of the column, *the unit stress for that load is multiplied by nineteen!* And this is the trifle that this defender and the writers of the Chicago Code consider too puerile to mention or to require any attention in the design, for their columns are designed for direct load only.

It is clear that the methods of design in reinforced concrete, as exhibited in building regulations and the Joint Committee Report, will not stand the search-light of logical analysis, else why do not those who are or ought to be capable of defending them bring forth a logical defense.

In conclusion, gentlemen, I am grateful for the privilege of delivering this paper. I have no desire to break down anything but error. I have no interest in any kind of building design other than an engineer's interest in seeing that designs are proper and safe. I have been plain and outspoken: anything else would be repugnant to me. I believe that the psychological moment has come for plain talk. The fact that I have been invited to read this paper indicates that others believe so also. It will not do for you to go away and treat these things with silence or sneers. It may be rare humor to some irresponsible author that he could ignore my criticism and let his book go to second edition with fatal errors. You who have the saving or sacrificing of lives had best ponder them.

To be presented at Chicago Convention—1915.
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NEW-OLD THEORY OF REINFORCED CONCRETE BEAMS IN BENDING.

By L. J. Mensch.*

Practically all ordinances of American and European building authorities insist on the following assumptions and requirements for the design of reinforced concrete slabs and beams:

1. Calculations should be made with reference to working stresses and safe loads, rather than with reference to ultimate strength and ultimate loads.

2. A plane section before bending remains plane after bending.

3. The steel to take all the direct tensile stresses.

4. The stress strain curve of concrete in compression to be assumed a straight line.

5. The ratio of the modulus of elasticity of the steel to the modulus of elasticity of the concrete, shall be taken at 15 for a concrete mixture of 1 : 2 : 4.

6. The extreme compressive stress of concrete in bending of 1 : 2 : 4 mixture to be from 600 to 750 lb. per sq. in.

7. The stress on mild steel in tension to be from 14,000 to 16,000 lb. per sq. in.

Computations according to above principles are not simple for slabs, are even complicated for T-beams and unreliable for nearly all other shapes. The greatest faults in the opinion of the author to be found with the above theory are that the results do not agree with actual tests, and it does not give to the designer any means of knowing what factor of safety his constructions has, and that it prevents a sound development of the art of reinforced concrete construction.

The following shows the factor of safety obtained by this theory for three different percentages of reinforcing:

Percentage of Reinforcement.	M/bd^2 figured.	M/bd^2 from tests.	Factor of Safety
½	73	209	2.85
1	118	386	3.27
2	140	648	4.63

*Contracting Engineer, Chicago, Ill.

These differences show that the accepted principles of designing reinforced concrete must be wrong, and, although great discrepancies between theory and test are found by nearly every experimenter, only very few have had the courage to discard, partially or in total, these theories which were evolved at the time when reinforced concrete was very little understood, and when only very few scientific tests on reinforced concrete beams had been made. Many investigators hold that the limit of resistance of a beam is reached when the tensile stress of the reinforcement

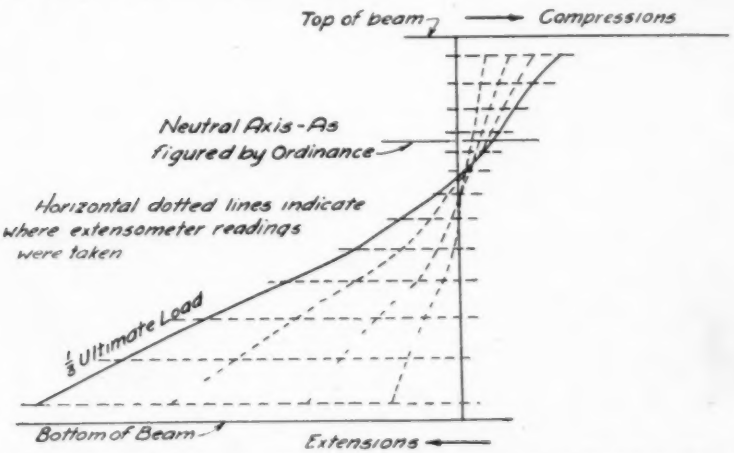


FIG. 1.—EXTENSOMETER READINGS ACROSS CRACK IN A REINFORCED CONCRETE BEAM.

reaches the yield point of the material, which for mild steel is about 39,000 lb. per sq. in., and yet they advocate a safe unit stress of 16,000 lb. per sq. in. on the tension reinforcement, evidently for no other reason than to obtain working formulas somewhat compatible with the results of their tests.

This theory is wrong because:

1. A plane section before bending is greatly distorted after bending. This was first discovered by Prof. Schule, of Zurich, in 1901, and later by other investigators by means of extensometer readings between horizontal points 2 to 6 in. apart and at 5 to 13 positions in a vertical line on the height of the beam, and they found that the extension of the steel fibres was a multiple of that which it should be on the plane section theory. Fig. 1 shows

the deformation of the compression and the reinforcement fibres in a concrete beam 8 in. deep, 15 ft. span, reinforced by 45 per cent mild steel, in a length of 2.4 in. measured across a crack in the beam near the center. Tests made by R. Miller of Leipsic, Germany.

2. The ratio of the moduli of elasticity is 10 to 20 when no crack appears in the concrete, but where a crack is present (which is practically identical with the assumption of allowing no tensile stress in the concrete) the ratio varies and may be as low as from 1 to 1/10 near the ultimate load. This latter point will be made clear by inspecting the stress-elongation diagram for mild steel shown in Fig. 2. An increase in stress of from 40,000 to 45,000 lb. produces an elongation of .02 in. corresponding to a modulus

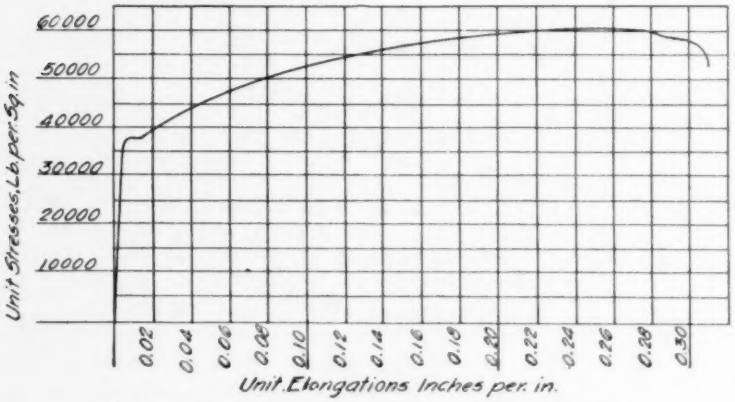


FIG. 2.—STRESS ELONGATION DIAGRAM FOR MILD STEEL.

of elasticity of $\frac{5000}{.02} = 250{,}000$ lb. per sq. in., while the modulus of elasticity of concrete in compression near the ultimate load varies from 2,000,000 to 3,000,000 lb.

3. The stress diagram for concrete in the compression part of a beam is in most cases a parabola and often approaches a rectangle or a trapezoid. In every report on the tension failures of tests of reinforced concrete beams, we find that the approaching failure was marked by vertical cracks gradually increasing in width and height, diminishing the compression zone, but only rarely the proper conclusion is added that the beam really failed finally by compression in the concrete and at a stress in the reinforcement

which is somewhat a mean of the elastic limit and the ultimate strength. Adopting the parabolic stress diagram for concrete near the ultimate load and an ultimate stress in mild steel reinforcement of 45,000 lb. per sq. in., and in commercial high carbon steel 72,000 lb. per sq. in., and in high carbon drawn steel wire of about 110,000 lb. per sq. in., and, of course, disregarding the tensile strength of the concrete, the author proposes to show that the formulas derived from the *equilibrium conditions at the moment of failure* are very simple and agree exceedingly well with tests made both here and abroad.

Let f_c = the ultimate strength of concrete in lb. per sq. in. as shown in cylinder tests.

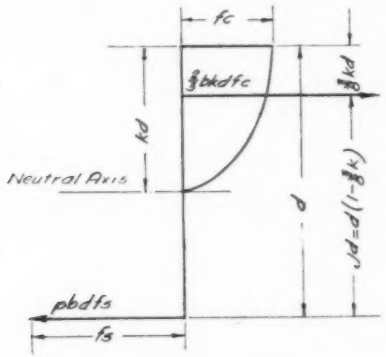


FIG. 3.

Let f_s = the ultimate stress of reinforcing steel in the sense as above given, and all other notations as given by the Joint Committee.

From Fig. 3: Total compression = $\frac{2}{3}bkdf_c$

Total tension = $pbdf_s$.

Compression = tension, or, $\frac{2}{3}bkdf_c = pbdf_s$

$k = p1.5f_s/f_c$ ..... (1)

$M = pbdf_s(d - \frac{3}{8}kd) = pbd^2f_s(1 - \frac{3}{8}k)$

or $M/bd^2 = pf_s(1 - \frac{3}{8}k)$ ..... (2)

If f_c is taken from cylinder tests, k and M can be immediately found. In Tables I and II the values of k and M/bd^2 for ultimate loads are given according to this theory for mild steel and high carbon steel reinforcement, respectively.

TABLE I.—ULTIMATE VALUES FOR M/bd^2 AND K FOR HIGH CARBON STEEL.

Ultimate Strength of Concrete.....	1000		1500		2000		2500		3000		3500		4000		4500		5000	
$K = p$ times.....	108		72		54		43.25		36		31		27		24		21.6	
Per cent of Steel.	M/bd^2	K	M/bd^2	K	M/bd^2	K	M/bd^2	K	M/bd^2	K	M/bd^2	K	M/bd^2	K	M/bd^2	K	M/bd^2	K
.25	162	.270	168	.180	171	.135	173	.108	174	.090	175	.078	175	.068	176	.060	176	.054
.30	190	.324	198	.216	203	.162	206	.130	207	.108	208	.093	210	.081	210	.072	211	.064
.35	216	.378	228	.252	234	.189	238	.151	240	.126	242	.108	243	.095	244	.084	245	.075
.40	242	.432	258	.288	264	.216	270	.173	272	.144	275	.124	277	.108	278	.096	279	.086
.45	265	.487	285	.324	294	.243	301	.195	305	.162	307	.140	310	.121	312	.108	313	.097
.50	287	.540	311	.360	323	.270	330	.216	336	.180	339	.155	342	.135	344	.120	345	.108
.55	308	.594	338	.396	352	.297	361	.238	366	.198	371	.171	374	.148	377	.134	379	.119
.60	327	.648	362	.432	380	.324	390	.260	397	.216	402	.186	406	.162	409	.144	412	.129
.65	345	.702	385	.468	406	.352	419	.281	426	.234	432	.201	438	.175	441	.156	443	.141
.70	361	.756	409	.504	433	.378	446	.303	456	.252	463	.217	468	.189	472	.168	475	.151
.75	377	.810	431	.540	457	.405	474	.325	485	.270	493	.233	499	.202	504	.180	506	.162
.80	390	.865	453	.576	484	.432	502	.346	515	.288	522	.248	530	.216	535	.192	539	.173
.85	...	....	472	.612	507	.459	528	.368	542	.306	552	.264	560	.230	566	.204	570	.184
.90	...	....	491	.648	530	.486	553	.389	570	.324	581	.279	589	.243	596	.216	601	.194
.95	...	....	509	.684	553	.513	580	.411	596	.342	609	.295	619	.256	625	.228	631	.205
1.00	...	....	526	.720	575	.540	603	.432	622	.360	636	.310	647	.270	655	.240	661	.216
1.10	...	....	558	.792	616	.594	651	.476	675	.396	691	.341	704	.297	713	.264	722	.238
1.20	...	....	585	.864	655	.648	696	.519	723	.432	748	.372	760	.324	770	.288	780	.259
1.30	...	....	...	....	690	.702	741	.562	773	.468	795	.403	815	.351	827	.312	838	.281
1.40	...	....	...	....	722	.756	780	.606	819	.504	845	.434	865	.378	882	.336	891	.310
1.50	...	....	...	....	753	.810	819	.649	863	.540	894	.465	915	.405	935	.360	950	.324
1.60	...	....	...	....	780	.864	857	.682	904	.576	939	.496	965	.432	990	.384	1004	.345
1.70	...	....	...	....	...	....	886	.736	945	.612	983	.527	1015	.459	1039	.408	1057	.367
1.80	...	....	...	....	...	....	920	.779	985	.648	1028	.558	1064	.486	1088	.432	1110	.389
1.90	...	....	...	....	...	....	946	.822	1018	.684	1065	.589	1105	.513	1132	.456	1158	.411
2.00	...	....	...	....	...	....	975	.865	1052	.720	1105	.620	1150	.540	1182	.480	1209	.432
2.25	...	....	...	....	...	....	...	....	1130	.810	1195	.698	1250	.608	1290	.540	1325	.486
2.50	...	....	...	....	...	....	...	....	....	....	1276	.775	1345	.675	1395	.600	1435	.540
2.75	...	....	...	....	...	....	...	....	....	....	1348	.853	1428	.743	1490	.660	1540	.596
3.00	...	....	...	....	...	....	...	....	....	....	....	....	1505	.810	1580	.720	1640	.648
3.50	...	....	...	....	...	....	...	....	....	....	....	....	....	....	1725	.840	1810	.755
4.00	...	....	...	....	...	....	...	....	....	....	....	....	....	....	....	....	1950	.864

TABLE II.—ULTIMATE VALUES FOR M/bd^2 AND K FOR MILD STEEL.

Ultimate Strength of Concrete	1000		1500		2000		2500		3000		3500		4000		4500		5000	
$K = p$ times	67.5		45		37.5		27		22.5		19.3		16.875		15		13.5	
Per cent of Steel.	M/bd^2	K	M/bd^2	K	M/bd^2	K	M/bd^2	K	M/bd^2	K	M/bd^2	K	M/bd^2	K	M/bd^2	K	M/bd^2	K
.25	105	.168	108	.112	108	.095	109	.067	110	.056	110	.048	111	.042	111	.038	111	.034
.30	124	.202	128	.135	129	.112	131	.081	132	.067	133	.058	133	.051	133	.045	133	.041
.35	143	.236	148	.1575	149	.132	152	.094	153	.079	153	.067	154	.059	154	.053	155	.047
.40	161	.270	168	.180	170	.150	173	.108	174	.090	175	.077	175	.068	176	.060	177	.054
.45	179	.304	187	.202	190	.168	193	.125	195	.102	196	.087	197	.076	197	.068	198	.061
.50	196	.337	206	.225	209	.187	213	.135	216	.1125	217	.096	218	.084	219	.075	219	.068
.55	213	.372	224	.247	228	.206	233	.148	236	.123	238	.106	239	.093	240	.083	241	.074
.60	229	.405	242	.270	247	.225	253	.162	256	.135	258	.116	260	.102	261	.090	261	.081
.65	245	.439	261	.292	266	.244	274	.176	277	.146	279	.126	281	.110	282	.098	283	.088
.70	259	.473	278	.315	284	.262	292	.189	296	.157	298	.135	301	.118	302	.105	303	.095
.75	273	.506	295	.337	302	.281	312	.202	315	.169	320	.145	322	.126	324	.113	325	.103
.80	287	.540	311	.360	319	.300	330	.216	335	.180	339	.155	341	135	344	.120	345	.108
.85	300	.574	328	.382	337	.319	349	.230	355	.191	359	.164	362	.143	364	.128	366	.115
.90	313	.608	343	.405	349	.337	368	.243	374	.202	379	.174	382	.152	384	.135	386	.122
.95	325	.642	358	.427	370	.356	386	.257	394	.214	399	.184	401	.160	406	.143	407	.128
1.00	338	.675	375	.450	386	.375	404	.270	412	.225	418	.193	422	.169	425	.150	427	.135
1.10	358	.742	403	.495	420	.412	440	.297	450	.247	455	.213	461	.186	465	.165	467	.149
1.20	376	.810	431	.540	449	.450	475	.324	485	.270	493	.232	498	.203	504	.180	506	.162
1.30	393	.878	457	.585	478	.487	507	.351	521	.292	530	.257	537	.219	542	.195	546	.176
1.40	...	....	481	.630	506	.525	540	.378	562	.315	565	.270	575	.236	580	.210	585	.189
1.50	...	....	505	.675	533	.562	573	.405	590	.337	602	.290	612	.253	617	.225	625	.202
1.60	...	....	526	.720	558	.600	603	.432	623	.360	637	.309	647	.270	655	.240	662	.216
1.70	...	....	546	.765	583	.637	635	.459	656	.382	671	.328	682	.287	693	.255	700	.230
1.80	...	....	565	.810	605	.675	663	.486	686	.405	705	.347	719	.304	728	.270	735	.243
1.90	...	....	580	.855	628	.712	690	.513	719	.427	736	.367	751	.321	756	.285	773	.251
2.00	...	....	...	....	648	.750	719	.540	748	.450	770	.386	784	.337	798	.300	809	.270
2.25	...	....	...	....	693	.843	782	.608	822	.505	850	.435	870	.380	885	.338	900	.304
2.50	...	....	...	....	...	....	840	.675	888	.562	920	.483	948	.422	966	.375	935	.338
2.75	...	....	...	....	...	....	894	.743	940	.617	990	.531	1025	.464	1047	.412	1065	.372
3.00	...	....	...	....	...	....	940	.810	1020	.675	1060	.580	1095	.506	1122	.450	1145	.405
3.50	...	....	...	....	...	....	...	....	1120	.784	1180	.675	1225	.590	1265	.525	1295	.473
4.00	...	....	...	....	...	....	...	....	1191	.900	1280	.772	1345	.674	1395	.600	1438	.540
4.50	...	....	...	....	...	....	...	....	....	....	1365	.868	1450	.758	1520	.675	1560	.607
5.00	...	....	...	....	...	....	...	....	....	....	....	....	1540	.842	1620	.750	1680	.675
5.50	...	....	...	....	...	....	...	....	....	....	....	....	....	....	1720	.825	1790	.742
6.00	...	....	...	....	...	....	...	....	....	....	....	....	....	....	....	....	1880	.810
6.50	...	....	...	....	...	....	...	....	....	....	....	....	....	....	....	....	1965	.878

The author wishes to repeat that the yield point in the reinforcing bars is considerably exceeded at the ultimate load, and, therefore, in beams having several layers of reinforcing bars, the reinforcement stress will vary only very little, although the bars are placed at different distances from the neutral axis, and this requires that the theoretical depth of such a beam should be assumed as the distance of the center of gravity of the reinforcing bars from the top of the beam.

We will now compare the ultimate coefficients of resistance given in the tables with the results of careful tests, made by well known investigators, where the failure was clearly a tension failure

TABLE III.—COMPARING TESTS BY MR. RICHARD L. HUMPHREY WITH TABLE II.

Nominal reinforcing, per cent	½	¾	1	1¼	1½	1¾	2
Number of ½-in. ϕ bars	2	3	4	5	6	7	8
Number of layers	1	1	1	2	2	2	2
Theoretical d	10.00	10.00	10.00	9.4	9.25	9.35	9.25
Actual per cent	0.49	0.74	0.98	1.30	1.59	1.84	2.13
fc for limestone concrete at 13 weeks	3600	3600	3600	3600	3600	3600	3600
Average ultimate load	6150	8640	11700	13260	15130	18060	20670
Max. M/bd^2 from test	208	284	375	476	560	647	753
Max. M/bd^2 from Table II	213	315	410	530	633	717	810
Variation, per cent	2.3	10.	8.5	10.	11.5	10.	7
fc for gravel concrete at 4 weeks	2567	2567	2567	2567	2567	2567	2567
Average ultimate load	6140	9220	12450	13760	15490	17890	20070
Max. M/bd^2 from test	208	301	397	494	572	640	733
Max. M/bd^2 from Table II	209	308	397	507	600	673	750
Variation, per cent	0	2.3	0	2.4	4.7	4.7	2.3
fc for cinder concrete at 26 weeks	2525	2525	2525	2525	2525	2525	2525
Average ultimate load	6460	9450	12810	13600	15320	17180	19810
Max. M/bd^2 from test	213	303	406	483	560	610	716
Max. M/bd^2 from Table II	209	308	397	507	600	673	750
Variation, per cent	1.9	1.6	2.3	4.7	7.	9.4	4.8

without suspicion of approaching bond or diagonal tension failure. Technologic Papers of the Bureau of Standard, No. 2, contains the result of tests made by Richard L. Humphrey on 333 beams, 8×11 in. in cross section, reinforced by percentages of mild steel varying from ½ to 2, 13 ft. long and tested on a 12-ft. span with symmetrical loads placed at ⅓ points. In some of the beams two layers of reinforcing bars were used, and, therefore, it will be necessary to use the distance of the center of gravity of the reinforcement from the top of the beam, in order to obtain the correct percentages of the metal.

Although cinder concrete has a modulus of elasticity of about $\frac{1}{3}$ that of gravel concrete, the cinder concrete beams were just as strong as those of gravel concrete of the same compressive strength, proving that the carrying capacity of a beam depends nearly entirely on the compressive strength and not on the value of n. There are very few test data published showing pure tension failures, and lack of time compels the author to confine the proofs to the tests he has at hand.

E. Mörsch, in his "Eisenbetonbau," 4th Edition, 1912, gives on page 167 a test on a beam of 80 in. span reinforced by 1.4 per cent of mild steel. The compressive strength of the concrete was exactly 2000 lb. per sq. in. and the beam failed at a $M/bd^2 = 528$, while the table gives $M/bd^2 = 506$.

In 1910, A. N. Talbot made tests on 8 × 12-in. beams, 6 ft. 6 in. long, tested by symmetrical loads at $\frac{1}{3}$ points of a 6-ft. span.

TABLE IV.—TESTS AT MASS. INST. TECH.

Beam.	3	4	5	6
Per cent of reinforcement	.31	.70	.96	1.25
Ultimate Moment (1000 in.-lb.)	268.4	470.5	549.3	580.8
M/bd^2 from test	335	587	686	725
M/bd^2 from Table I	213	456	601	748

The beams were reinforced by 1.61 per cent of high carbon corrugated bars, portions of which were bent up and connected to a vertical and inclined stirrup system. The center of gravity of the reinforcement was 10 in. below the top of the beam. The beams 284.1, 284.3, 284.5, 284.6, 284.7, failed at an average ultimate load of 52,400 lb., or a moment of $\frac{52400}{2} \times 24 = 628{,}800$ in.-lb., or $M/bd^2 = 786$, while the table for 2000 lb. concrete gives 780.

In Bulletin No. 29 of the University of Illinois, entitled *Test of Reinforced Concrete Beams; Resistance to Webstresses*, on page 63, beam No. 223.1, 8×11 in., reinforced by 1.25 per cent of high carbon steel, span 6 ft., loaded at $\frac{1}{3}$ points, failed in tension at a load of 43,300 lb. corresponding to a moment of $\frac{43{,}300}{2} \times 24 =$ 519,600 in.-lb., or $M/bd^2 = 649.5$, while the table gives 670.

At the Massachusetts Institute of Technology, tests were made in 1901 on beams 8 × 12 in. in section, on a 11-ft. span, loaded at the $\frac{1}{3}$ points. The reinforcing consisted of twisted bars, and varied from .31 to 3.9 per cent; only a few tension failures were recorded and are shown in Table IV. It is clear that the trap rock concrete must have been of excellent quality, and it is probable that the smaller sized twisted bars had a much higher yield point and ultimate strength than ordinary high carbon bars.

Comparing the two tables we find that for the same percentage of reinforcement and for the same compressive strength of concrete the ultimate M/bd^2 is over 50 per cent greater for high carbon steel than for medium steel. All specifications calling for the use of medium steel invite, therefore, a considerable waste of money, and granting even that high carbon steel is not as reliable as mild steel, the chances are very remote that $\frac{1}{3}$ of the steel rods are faulty and that all faults should occur at the same point in the beam. The old theory let us believe that beams reinforced by more than 1 per cent of high carbon steel or $1\frac{1}{2}$ per cent of mild steel, failed in compression and that the steel stress at failure did not reach even the elastic limit. The new theory claims that all beams designed according to Tables I and II will fail in compression and tension simultaneously, although the deformation of the steel fibres is much greater than that of the compression fibres, making observers liable to call all failures tension failures.

When the reinforcement much exceeds the percentages given in the lower portion of the tables, the steel will not reach any more the stresses of 45,000 or 72,000 lb. at the time of failure, and we will have a true compression failure, viz: a failure of a concrete beam where nearly the entire portion of the beam above the tension reinforcement is in compression and where crushing of the compression part will take place before the steel fibres indicate approaching failures. The limiting amount of reinforcing between a simultaneous compression and tension failure and a true compression failure, we call the balanced reinforcement, and we claim that the carrying capacity of a beam cannot be increased by increasing this balanced reinforcement, as such an increase has only the effect to decrease the steel stress, but cannot in the nature of the case increase f_c nor can it increase kd because we claim balanced

reinforcement is reached when $kd=d$, and the maximum possible value, as shown in Fig. 4—From equation (1) $k=p\ 1.5\ f_s/f_c$ here $k=1$.

Hence the balanced reinforcement $p=f_c/1.5f_s$.....(3)

And $M=\frac{2}{3}bdf_c\times\frac{5}{8}d=\frac{10}{24}bd^2f_c$, or $M/bd^2=f_c/2.4$.....(4)

Prof. Mörsch made a companion test to that previously mentioned, which differed only in the percentage of the reinforcement, the latter being 3.3 per cent. The moment at failure was 97,000 in.-lb.; $b=9.8$ in., $d=3.55$ in., $M/bd^2=784$ while according to formula (4), $f_c/2.4=\dfrac{2000}{2.4}=833$.

It may be expected that formula (4) gives higher values, because k really can never reach the value 1.

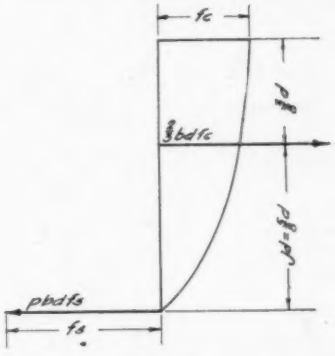


FIG. 4.

We can obtain a better agreement with tests by assuming for balanced reinforcement $M/bd^2=f_c/2.6$.....(5)

Which corresponds to $k=.84$ and a balanced reinforcement $p=f_c/1.78\ f_s$.....(6)

For this rule in the above case $M/bd^2=780$.

Prof. Mörsch also mentions on page 172 of his book, tests made by Prof. Bach, of Stuttgart, on beams reinforced by 7.1 per cent of concrete having a compressive strength of only 1620 lb. per sq. in. The balanced reinforcement in the case would be according to formula (6) only $\dfrac{1620}{1.78}\times 45{,}000=.02$, and if our theory is correct, this highly over-reinforced beam should not carry any

more than if reinforced by only 2 per cent; and indeed we find that this beam failed at an ultimate $M/bd^2 = 644$, while according to formula (5) for balanced reinforcement $M/bd^2 = \frac{1620}{2.6} = 680$.

Replacing in formula (5) f_c by the tensile strength of the concrete f_t, we obtain an expression for the ultimate bending moment of a plain concrete beam $M = bd^2 f_t/2.6$ ..... (7)

While the commonly quoted expression for this moment, using the so-called modulus of rupture S (which should be the tensile strength of the concrete) is $M = bd^2S/2.6$; on the other we know that for a good grade of concrete $M/bd^2 = 100$, hence $f_t = 260$ lb. per sq. in., and $S = 600$ lb. per sq. in.

Inasmuch as the value f_t obtained by formula (7) agrees with

TABLE V.

Beam.	Reinforcement. Top. Bottom.	Ultimate Movement (1000 in.-lbs.)	Increase of Ultimate Movement over "a." (1000 in.-lb.)	Unit Stress in Top Reinforcing.
a	none 4-3 cm. ϕ mild steel	342.5	0	.....
b	2-1 cm. ϕ mild steel 4-3 cm. ϕ mild steel	419.5	77	41500
d and e	2-2 cm. ϕ mild steel 4-3 cm. ϕ mild steel	577.5	235	31500
f	2-2 cm. ϕ high carbon 4-3 cm. ϕ mild steel	717.5	375	50400

direct tension tests, formula (7) should be accepted as a proper rule for the design of plain concrete beams.

We will now further check our theory by tests made on reinforced concrete beams, having also compressive reinforcement. Inspecting again Fig. 1, we find that after mild steel is stressed beyond the yield point, the modulus of elasticity decreases very rapidly, and n approaches the value 1, very soon after the yield point is acceded. Therefore, we cannot expect that compression steel will take up at the ultimate load a higher stress than the yield point of the material. This is clearly shown by the tests, mentioned by Prof. Mörsch, on page 174 of his book on beams reinforced by 7.1 per cent in tension and by $\frac{3}{8}$-in. and $\frac{3}{4}$-in. bars of mild and high carbon steel in compression as shown in Table V.

Often the bars of small diameter have a higher yield point, which explains the higher steel stress in beams *b* than in beams *d* and *e*. It also shows that we can safely figure for mild steel on an ultimate stress in compression of 32,000, and for high carbon steel of 50,000 lb. per sq. in.

Bearing in mind that the mild steel reinforcing at the time of failure is stressed to about 32,000 lb., we obtain from Fig. 5 the following equilibrium conditions for the beams reinforced also on the compressive side at the ultimate load,

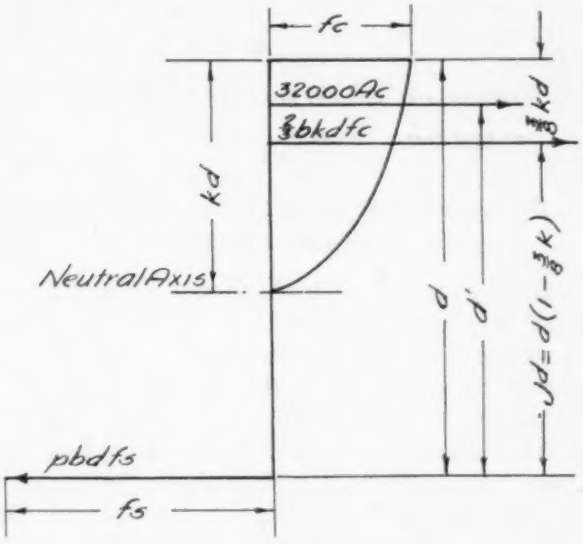


FIG. 5.

$$\tfrac{2}{3} b k d f_c + Ac\,32{,}000 = p b d f_s$$

$$\text{or, } k = 1.5\, f_s/f_c - 48{,}000\ Ac/f_c\, bd \ldots\ldots\ldots (8)$$

$$\text{and } M = \tfrac{2}{3} f_c\, b k d\, (d - \tfrac{3}{8} kd) + 32{,}000\ Ac\, d' \ldots . (9)$$

In Bulletin No. 2, Vol. 4, Engineering Series, University of Wisconsin, M. O. Withey gives on page 89 the results of 4 beams reinforced by 2.9 per cent on the tension side, and by 1 and 1½ per cent of mild steel on the compression side. Beams *W* failed at an average ultimate moment of 814,500 in.-lb., and their concrete test cylinders failed at 2,600 lb. per sq., in.

$$Ac = .784 \text{ sq. in.}, f_s = 45{,}000 \text{ lb.}, f_c = 2600 \text{ lb.}$$
$$b = 8 \text{ in.}, d = 9\tfrac{3}{4} \text{ in.}, d' = 8\tfrac{1}{2} \text{ in.}$$

From formulas (8) and (9) we have

$$k = 1.5 \times \frac{45{,}000}{2600}\,\frac{2.9}{100} - \frac{48{,}000}{2600}\,\frac{784}{8 \times 9\frac{3}{4}} = .753 - .185 = .568$$

$$\text{and } M = \tfrac{2}{3}\, 2600 \times 8 \times 9\tfrac{3}{4}^2 \times .568\,(1 - .213) + 32{,}000 \times .784 \times 8\tfrac{1}{2}$$
$$= 588{,}000 + 213{,}000 = 801000 \text{ in.-lb.}$$

showing a very close agreement.

Beams X had $1\frac{1}{2}$ per cent top reinforcement and were of poorer concrete, having an average compressive strength of only 1515 lb. per sq. in.

From formulas (8) and (9) we have again

$$k = 1.5 \times \frac{45{,}000}{1515}\,\frac{2.9}{100} - \frac{48{,}000}{1515} \times \frac{1.176}{8 \times 9\frac{3}{4}} = 1.293 - .477 = .816$$

$$\text{and } M = \tfrac{2}{3} \times 1515 \times 8 \times 9\tfrac{3}{4}^2 \times .816(1 - .306) + 32{,}000 \times 1.176 \times 8\tfrac{1}{2} =$$
$$433{,}000 + 320{,}000 = 753{,}000 \text{ in.-lb.}$$

while the average moment of both beams is 808,000 in.-lb., or a difference of only 7 per cent.

In regard to the design of *T* beams, the latest European tests made by Prof. Bach in behalf of the German *Jubiläum's Stiftung*, indicate that we can safely count on the flange acting together with the stem, when the width of the flange is less than about $\frac{1}{3}$ of the span of the beam and when the projection on one side is less than 7 times the slab thickness; only for such high ratios of projection to flange thickness it is absolutely necessary to provide reinforcement in the slab at right angle to the span. Where the value of *kd* of the inclosing rectangle of flange and stem is less than the thickness of the flange, our tables directly apply; for larger value of *kd*, it is common and safe practice to assume that the compressive stress is uniformly distributed over the flange and that the ultimate load may be found by multiplying the entire tension (steel area times 45,000 and 72,000 lb., respectively) by the distance from the center of reinforcing to the center of flange.

It must be borne in mind that this theory assumes only pure tension and compression failures, which as a rule can be always

counted upon in beams, where the shearing values at the ultimate load are not larger than about 150 lb. per sq. in. Where the ultimate shear values much exceed 150 lb., it is absolutely necessary to make ample provision to overcome premature bond and diagonal tension failures; this is best done by bending up $\frac{1}{2}$ the number of bars in various planes and by using an ample number of stirrups; and in case of non-continuous beams, by providing large hooks at the end of the bars.

Where a high percentage of reinforcement is used, the table shows that there is quite a great difference in the carrying power of the beam for the various grades of concrete, and the author advises not to count a higher ultimate strength, under expert supervision, than

2000	lb.	on	well	graded	1 : 2 : 4	concrete,
3000	"	"	"	"	1 : $1\frac{1}{2}$: 3	"
4000	"	"	"	"	1 : 1 : 2	"
5000	"	"	"	"	1 : 1 : 1	"

while 25 per cent less should be used where the concrete materials are not so well graded.

INSTITUTE NOTES.

Committee on Edison Fire.

It was deemed highly desirable on account of the unusual conditions surrounding the fire in the plant of T. A. Edison, Inc., at West Orange, N. J., on December 9, 1914, to appoint a Committee to make a special investigation for the Institute. The following Committee was appointed by the President:

CASS GILBERT, *Chairman*,	RICHARD L. HUMPHREY,
E. J. MOORE, *Secretary*.	RUDOLPH P. MILLER,
WALTER COOK,	CHARLES L. NORTON,
WILLIAM H. HAM,	J. KNOX TAYLOR,

This Committee has visited the plant, held a number of meetings and planned a careful investigation, having as its special object the study of the behavior of concrete and reinforced concrete in this conflagration. The Committee, in co-operation with T. A. Edison, Inc., has arranged special investigations and tests. Inasmuch as these will require some time, the Committee deems it desirable to present a preliminary report at the Chicago Convention and subsequently a final report which will appear in the JOURNAL.

Result of Letter-Ballot on By-Laws and Specifications.

The letter-ballot canvassed on December 10, 1914, resulted in the adoption of the following items as printed in the JOURNAL, Vol. II, No. 6, pp. 5–46:

Amendments to By-Laws.

Standard Specifications for One-Course Concrete Highway (Standard No. 5).

Standard Specifications for One-Course Concrete Street Pavement (Standard No. 17).

Standard Specifications for Two-Course Concrete Street Pavement (Standard No. 18).

Standard Specifications for Portland Cement Stucco on Metal Lath, Brick, Tile or Concrete Block (Standard No. 15).

Standards Adopted by the Institute.

1. Standard Specifications for Cement with Appendix on Standard Methods of Testing and Chemical Analysis, 23 pp.
2. Standard Specifications for Portland Cement Sidewalks, 6 pp.
4. Standard Building Regulations for the Use of Reinforced Concrete, 13 pp.
5. Standard Specifications for One Course Concrete Highway, 10 pp.
6. Standard Specifications for Portland Cement Curb and Gutter, 7 pp.
7. Standard Recommended Practice for the Use of Reinforced Concrete, 21 pp.
8. Standard Specifications for Scrubbed Concrete Surfaces, 3 pp.
9. Standard Recommended Practice for Concrete Drain Tile, 3 pp.
10. Standard Recommended Practice for Concrete Architectural Stone, Building Block and Brick, 4 pp.
11. Standard Specifications for Concrete Architectural Stone, Building Block and Brick, 3 pp.
12. Standard Building Regulations for the Use of Concrete Architectural Stone, Building Block and Brick, 3 pp.
13. Standard Specifications for Plain Concrete Floors, 6 pp.
14. Standard Specifications for Reinforced Concrete Floors, 5 pp.
15. Standard Specifications for Portland Cement Stucco on Metal Lath, Brick, Tile or Concrete Block, 9 pp.
16. Standard Methods for the Measurement of Concrete Work, 7 pp.
17. Standard Specifications for One Course Concrete Street Pavement, 10 pp.
18. Standard Specifications for Two Course Concrete Street Pavement, 11 pp.

The above Standards can be obtained at the price of 25 cents each or in lots of ten or more at 20 cents each. Special price to Members 15 cents each or 10 cents each in lots of ten.

Reduced Prices for Back Volumes.

Members of the Institute can obtain copies of back publications at the following special prices:

Vol. I, 50 cents; Vols. II to VI, $1.50 each; all in paper binding. For cloth or half morocco binding there is an additional charge per volume of 50 cents or $1.00 respectively. Vol. VII is available only in cloth binding, $2.00, or half morocco, $2.50.

New Members.

The following new members have been elected:

Billings, A. W. K. Vice-President and Managing Director, Ebro Irrigation and Power Company, Ltd., Apartado 491, Barcelona, Spain.

Bone, Frank A. Engineer and Contractor, 2633 Cleinview Avenue, Cincinnati, Ohio.

Cook, Fred W. Concrete Contractor, 136 North Alabama Avenue, Okmulgee, Okla.

Coombs, Loren Robert. Superintendent of Construction, State of Washington, 3725 South L Street, Tacoma, Washington.

Jenks, Harvey E. Superintendent of Construction, Norfolk State Hospital, Box A, Wrentham, Mass.

Johnson, Nathan C. Engineer of Tests, Raymond Concrete Pile Company, 140 Cedar Street, New York, N. Y.

Kikuchi, Aitaro. Chemist, Darien Branch, Onoda Cement Manufacturing Company, Box 24, Darien, Dalny, Manchuria, China.

Morrow, Clarence E. Instructor in Architectural Engineering, Massachusetts Institute of Technology, H. L. Pierce Building, 30 Trinity Place, Boston, Mass.

Nelson and Witt. Cement Products, A. P. Nelson, General Manager, West First Street, Spencer, Iowa.

Pagon, William Watters. Engineer, Creiner and Whitman, 173 Washington Street, South Norwalk, Conn.

Peabody, Jr., Dean. Instructor, Massachusetts Institute of Technology, 85 Bartlett Street, Somerville, Mass.

Sears, Robert S. Contractor, Flushing, Ohio.

Security Cement and Lime Company. John J. Porter, General Manager, Hagerstown, Md.

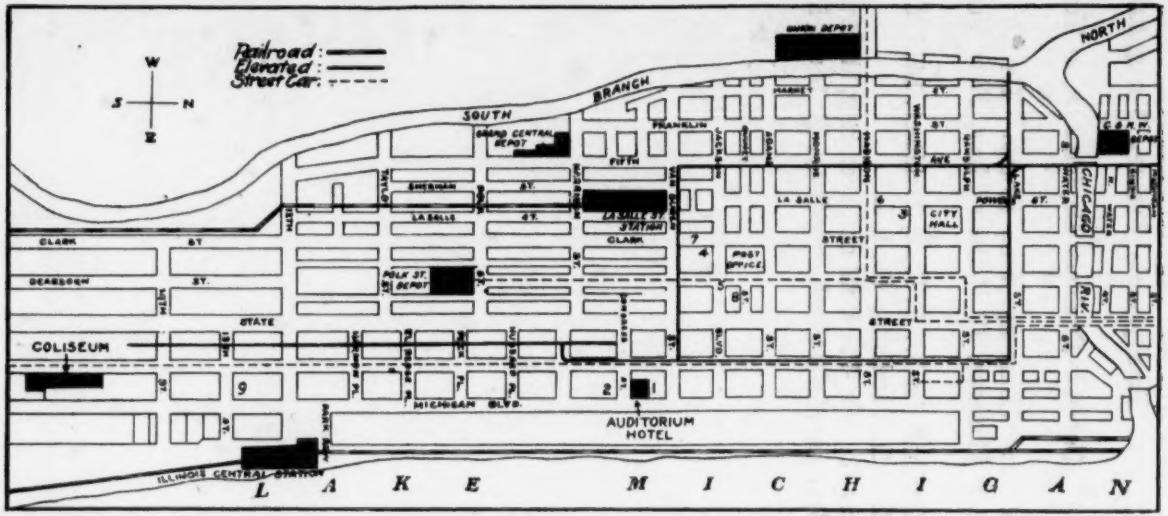
Snodgrass, Robert Davis. Chief Engineer, Trussed Concrete Steel Company, 841 Pennsylvania Avenue, Youngstown, Ohio.

Staples, Harry John. Concrete Products, Dayton Avenue, Toms River, N. J.

Steers Sand and Gravel Company, Inc., Henry. Harold H. Seeley, Assistant Manager, 17 Battery Place, New York, N. Y.

Weiss, Frank J. Engineer for W. E. Russ, Architect, 1405 Merchants' Bank Building, Indianapolis, Ind.

Whitman, Jr., Henry L. Vice-President and Secretary, Whitman Agricultural Company, 6900 South Broadway, St. Louis, Mo.



PORTION OF CITY OF CHICAGO SHOWING LOCATION OF CONVENTION AND EXHIBITION HALLS, PRINCIPAL DEPOTS AND HOTELS.

KEY

1 AUDITORIUM HOTEL
2 CONGRESS HOTEL
3 HOTEL SHERMAN
4 GRAND PACIFIC HOTEL
5 GREAT NORTHERN HOTEL
6 HOTEL LA SALLE
7 KAISERHOF
8 LEXINGTON HOTEL
9 NEW SOUTHERN HOTEL

Synopses of Recent Articles on Concrete.

Reviewed November 1 to 30, 1914.

The synopses given below are for the purpose of keeping the members of the Institute in touch with the leading articles of interest appearing in the many technical journals and is not a complete review of the journal. The publications regularly reviewed appear below and the number given corresponds to the number following the title of the paper. The length of the article in pages, whether illustrated or not, is noted, and the publications reviewed in which it appears:

1. Canadian Engineer, Toronto, Ontario, Canada.
2. Cement, New York, N. Y.
3. Cement Era, Chicago, Ill.
4. Cement World, Chicago, Ill.
5. Concrete Age, Atlanta, Ga.
6. Concrete Era, Los Angeles, Cal.
7. Concrete-Cement Age, Detroit, Mich.
8. Concrete and Constructional Engineering, London, England.
9. Engineering and Contracting, Chicago, Ill.
10. Engineering News, New York, N. Y.
11. Engineering Record, New York, N. Y.
12. Good Roads Magazine, New York, N. Y.
13. Portland Cement, Kansas City, Mo.
14. Rock Products, Chicago, Ill.
15. Proceedings American Railway Engineering Association, Chicago, Ill.
16. Western Contractor, Record Building, Kansas City, Mo.
17. Journal National Fire Protection Association, Boston, Mass.
18. Journal Western Society of Engineers, Chicago, Ill.
19. Proceedings American Society of Civil Engineers, New York, N. Y.
20. Proceedings American Society of Municipal Improvements, New York.
21. Proceedings American Society for Testing Materials, Philadelphia, Pa.
22. Proceedings Engineers' Club of Philadelphia, Philadelphia, Pa.
23. Proceedings Engineers' Society of Western Pennsylvania, Pittsburgh, Pa.
24. Professional Memoirs, Corps of Engrs., U. S. A., Washington, D. C.
25. Proceedings Institution of Civil Engineers, London, England.
26. Transactions, Canadian Society of Civil Engineers, Montreal, Quebec, Canada.
27. Railway Age Gazette, Chicago, Ill.
28. Better Roads, Jamestown, Ohio.

MATERIALS

Development of Sand and Gravel Deposits, by W. H. Wilms. (**10**) Nov. 5, 3½ pp., ill.; Nov. 12, 4½ pp., ill.; Nov. 19, 5 pp., ill. Description of the type of steam shovel to use; the essential points in stripping overburden by hydraulicing; the layout, grades and construction features of railway tracks; crushing, storage and power plants.

Proposed Standard Abrasion Test for Gravel. (**10**) Nov. 5, ½ p. Result of an investigation conducted by the Bureau of Tests, Ohio State Highway Department.

Permeability of Gravel Concrete. (**1**) Nov. 5, ½ pp. Summary of tests at University of Wisconsin showing effects produced by age, mixtures, curing, grading of aggregates, etc.

DESIGN AND TESTS

Concrete Tower for Water Tank, Erie R. R. (**10**) Nov. 12, ½ p., ill. Details of 50,000-gallon water tank.

BUILDINGS

Structural Features of Reinforced Concrete Combined Storage Warehouse and Office Building, in Seattle, Wash. (**9**) Nov. 11, 4½ pp., ill. Design and construction of building 436 ft. x 80 ft., 5 stories and basement.

Rapid Construction of a Large Reinforced-Concrete Warehouse. (**10**) Nov. 12, 1½ pp., ill. Description of warehouse and details of concrete plant on Austin-Nichols plant, Brooklyn, N. Y.

Construction Methods on New Concrete Buildings for M. I. T. in Cambridge. (**11**) Nov. 28, 3½ pp., ill. Description of plants used for concreting 3¼ acres of structures. Systematic field tests of concrete.

Concrete Stadium at Michigan. (**4**) Nov., 1½ pp., ill. Details of reinforced concrete grandstand at the University of Michigan.

Report on a Series of Tests on Concrete Columns Reinforced with a Spiral of Steel, by C. G. Wrentmore, (**19**) Nov., 3½ pp.

The Use of Concrete in the Substructure of the New County Hall, London. (**8**) Nov., 8½ pp., ill. Building 750 x 350 ft. in plan, 128 ft. high, on a concrete raft consisting of five 12-inch layers of concrete.

Reinforced Concrete Buildings. (**6**) Nov., 2 pp., ill. Hollow-wall system entirely around house; pouring height of wall of 2 ft. for entire structure at one time.

Notes on Surface Finish of Concrete Factory Building, Hartford, Conn. (**7**) Nov., 1 p. Factory building 52 ft. x 213 ft., 4-story and basement.

Unit-Built Reinforced Concrete Way-Stations, Pacific Electric Railway, Los Angeles, Cal. (**7**) Nov., 2 pp., ill. Shelters 8 ft. x 12 ft., 8 ft. in the clear, consisting of 119 units.

HOMES

Concrete and Philanthropy. (**4**) Nov., 2 pp., ill. Description of modern homes for working men.

Our Concrete Cottage Competition. (**8**) Nov., 12 pp., ill. Plans, specifications and estimated costs.

How to Erect Small Concrete Buildings. (**13**) Nov., 2 pp.

BRIDGES AND RAILROAD WORK

Design and Construction of the San Jacinto Street Reinforced Concrete Bridge, Houston, Texas. (**9**) Nov. 25, 3 pp., ill. General description of design, construction features, forms and centering and placing of reinforcement.

Concrete Viaduct with 50-ft. Walls. (**10**) Nov. 5, 2½ pp., ill. Design and construction of viaduct carrying Union Ave. over tracks of the St. Louis Terminal R. R.

Slate Creek Bridge: Building Concrete Arch Bridge 56 Miles from Rail, by Jean H. Knox. (**10**) Nov. 19, 2½ pp., ill. 66-ft. reinforced concrete arch replacing wood and steel bridge in Sierra Nevada Mountains, Cal. Concrete bridge cheaper than steel structure.

Conveying Concrete Materials by Belts at the Hell Gate Bridge. (**10**) Nov. 26, 1 p., ill. Description of method of handling cement, sand, gravel and stone from barge to storage bin and pile.

Construction of the New York Connecting R. R. (**27**) Nov. 13, 4½ pp., ill. Description of ten-mile line, involving large bridges and viaducts to connect the Pennsylvania and New Haven Roads.

Concrete Highway Bridge Construction as Standardized by Iowa Commission, by C. B. McCullough. (**11**) Nov. 7, 3½ pp., ill.

Large Trussed Concrete Bridge. (**11**) Nov. 14, ½ pp., ill. Description of structure, 182-ft. span, with parabolic reinforced-concrete compression chord, believed to be the longest of its kind.

"Low-Water" Concrete Bridges, by W. S. Gearheart, State Engineer of Kansas. (**4**) Nov. 2½ pp., ill. A new form of reinforced concrete structure designed to withstand the severest floods.

History of Little Rock Junction Rwy. Bridge, of the St. Louis, Iron Mountain and Southern Rwy. Co., over the Arkansas River at Little Rock, Ark., 1883–1914, by C. E. Smith, (**19**) Nov., 105 pp., ill. Account of the reconstruction of old piers.

What the B. & O. Has Done With Concrete. (**6**) Nov., 3 pp. Review of the improvements made during the last four years.

New Highway Bridge in Mason City. (6) Nov., ½ p. Two 55-ft. arch spans to be erected by Iowa Highway Commission, combining art and utility.

Concrete Replacing Wooden Bridges. (6) Nov., 1 p., ill. Concrete bridges and culverts at McAlester, Okla., replacing wooden structures.

RESERVOIRS, DAMS, CANALS, ETC.

Lesson Taught by the Construction, Failure and Reconstruction of a Small Dam Near Tullahoma, Tenn. (9) Nov. 11, 2½ pp. Discussion of the engineer's responsibility for works of his design and construction and of the ethics of cases wherein engineers do not supervise the construction of works designed by them.

Construction Plant Layout for New Filtration Plant at Cleveland, Ohio. (10) Nov. 5, 2 pp., ill. Description and details of concrete mixing plant.

A Government Power Dam for Municipal Use. (10) Nov. 12, 3 pp., ill. Government dam project in Mississippi changed (when half built) to afford a power development.

New Dry Dock for San Francisco. (10) Nov. 26, 2 pp., ill. The third largest dry dock in the world now under construction.

A Small Reinforced Concrete Dam with a Large Upstream Apron of Concrete Paving. (9) Nov. 26, 1½ pp., ill. A 50-acre concrete mat placed on the porus bank of a small private reservoir.

Construction Plant for Cleveland Filters. (11) Nov. 7, 2 pp., ill. Description of elaborate equipment enabling contractor to place 18,000 cu. yds. concrete per month, and gain 5 months on schedule time.

West Side Tunnel Extension at Cleveland. (11) Nov. 21, 2½ pp., ill. Description of three-mile, ten-foot tunnel driven by a special excavator and lined with huge concrete blocks set by a hydraulic erector.

The Year's Work on the Panama Canal. (11) Nov. 21, 5 pp., ill. An abstract of annual report by Governor Goethals, covering construction, operation and maintenance for fiscal year ended June 30, 1914.

Walnut Hill Reservoir, Omaha, Neb. (13) Nov., 1 p. Basin has capacity of 11,000,000 gallons, is 359 ft. long x 277 ft. wide at high water line, mean depth about 25.5 ft.

Big Concrete Unit for Water Supply. (6) Nov., 2 pp., ill. Hill View Reservoir, New York City, method of construction.

ROADS AND PAVEMENTS

Construction and Maintenance Details of Concrete Roadwork in Wayne County, Mich., in 1914. (9) Nov. 11, 2½ pp., ill. Adaptability and cost of concrete macadam pavement bases in Oakland, Cal.

Cost of Concrete Culverts and Bridges in Milwaukee, Wis., in 1913. (9) Nov. 18, 1 p., ill.

Rebuilding and Surfacing the Old National Pike with a Concrete Pavement. (10) Nov. 5, 1½ p., ill. History and present condition of old pike—new work.

How California is Building an $18,000,000 System of State Highways, by Austin B. Fletcher, (11) Nov. 7, 2 pp., ill. Account of many economies—Feature of work is successful use of 4-inch concrete base with thin bituminous top.

New Concrete Road on the Way South. (6) Nov., 1¼ pp. Road building in Kentucky.

Construction and Cost Data on Concrete Paving in Macon, Ga., by Carl H. Fuller. (7) Nov., 1½ pp. As an experiment, a short section paved with concrete; smoth surface and low cost features of instant popularity.

Concrete Road Built at Montreal to Test Different Types of Construction. (7) Nov., 2 pp., ill. A concrete road 2510 ft. long and 14 ft. wide built in 15 sections varying in length from 19 to 394 ft.

SEWERS, PIPE, TILE, ETC.

Converting Old Septic Tank and Contact Beds into Two-story Tank and Sprinkling Filters at Moorestown, N. J.—Operating Results. (9) Nov. 18, 3 pp., ill. Descriptions of original and remodeled plants.

Reconstruction of Pogues Run at Indianapolis. (10) Nov. 5, 1 pp., ill. Description of reinforced concrete conduit.

New York Rapid Transit Rwy. Extensions, by F. Lavis. (10) Nov. 12, 7 pp., ill. Description of sewers, pipes, conduits, etc.

Enlarging an Old Egg-shaped Brick Sewer, by G. L. Christian. (11) Nov. 7, 1 p., ill. Description of utilization of existing invert and use of reinforced concrete to widen section at top—Details of collapsible forms.

Concrete Pipe for Big Sewer. (6) Nov., 1 p. 25 miles of 10- to 60-in. diam. concrete pipe for Salt Lake City, Utah.

Big Trunk Sewer Built of Concrete. (6) Nov., 3 pp., ill. New concrete sewer 6 x 9 ft., egg-shaped, for Chicago system.

CONCRETE PRODUCTS

Cement Tile Roofing. (27) Nov. 20, 1 p., ill. Account of large installation of cement tile in new Union Station at Kansas City, Mo.

Concrete Poles in San Francisco, by Clarence P. Kane. (4) Nov., 3½ pp., ill. Description of artistic concrete electric light and power poles.

MISCELLANEOUS

Design and Cost of Two Types of Concrete and Metal Fence. (**9**) Nov. 18, ½ p., ill.

The Yale "Bowl"; An Amphitheater of Earthwork, by Thomas C. Attwood. (**10**) Nov. 12, 5 pp., ill. Description and design of colosseum at New Haven, seating 61,000 spectators.

Concrete Lining for Steel Bunkers. (**27**) Nov. 20, 1 p., ill. Account of successful lining of steel coal bunker by the use of the cement gun.

Bulk-Cement Handling Plant, Northwest Station, Commonwealth Edison Co., Chicago, by by Harold W. Robbins. (**10**) Nov. 26, 2½ pp., ill. Method of elevating bulk cement to bins and chuting to mixer; effective and economical.

Concrete Highway Signs. (**27**) Nov. 20, ¼ p. Account of the durability of these signs.

Yale "Bowl" Construction Finished in Time for Harvard Game To-day. (**11**) Nov. 21, 3 pp., ill. Account of construction.

Steel Towers in Concrete Distribution, by P. C. Ferguson. (**4**) Nov., 2¼ pp., ill. Details of construction and operation by modern hoisting and chuting equipment.

Models and Molds in Making Ornamental Concrete Castings. (**7**) Nov., ½ p., ill.

PART II. Papers, Discussions and Reports
(Forming part of Vol. X of *Proceedings*.)

DATA ON LIME PUTTY AND CREAM OF LIME.

By Cloyd M. Chapman.*

(*Concluded.*)

The points all fall very close to a curve which has been drawn in with a full line. This curve conforms to the formula—

$$y = \frac{200}{\frac{x}{7.5} \quad .8}$$ in which

y = gallons of water added to the barrel of lime.
x = lb. of lime in 1 cu. ft. of the resulting mixture.
200 = lb. of lime in 1 bbl.
7.5 = gals. of water in 1 cu. ft.
.8 = approximate absolute volume in cu. ft. of 200 lb. of lime.

It is apparent therefore from the close agreement between the points determined experimentally and the mathematically plotted curve that it is quite unnecessary to perform any experimental work in the laboratory to determine the amount of lime in a given volume of a lime putty of any consistency, provided the amount of water that has been mixed with a given weight of lime is known. By the use of the curve or the above formula the lb. of lime in a cu. ft. of any mixture of lime and water may readily be determined. The difficulty lies in knowing, or being able to determine, the amount of water that is actually added to the lump lime. This difficulty is brought about by the evaporation of a considerable portion of the water that is added to a lump lime during the slaking process. Even if the water added to a lump is carefully measured, there is no easy method of determining on the job how much water is left after slaking is complete. A series of trials made in the laboratory with all materials at room temperature of about 70 deg. F. showed an average evaporation of about 20 per cent of the water. This would

* Engineer Tests, Westinghouse Church Kerr and Company, New York, N. Y.

probably vary a great deal, depending upon conditions of temperature and amount of water used.

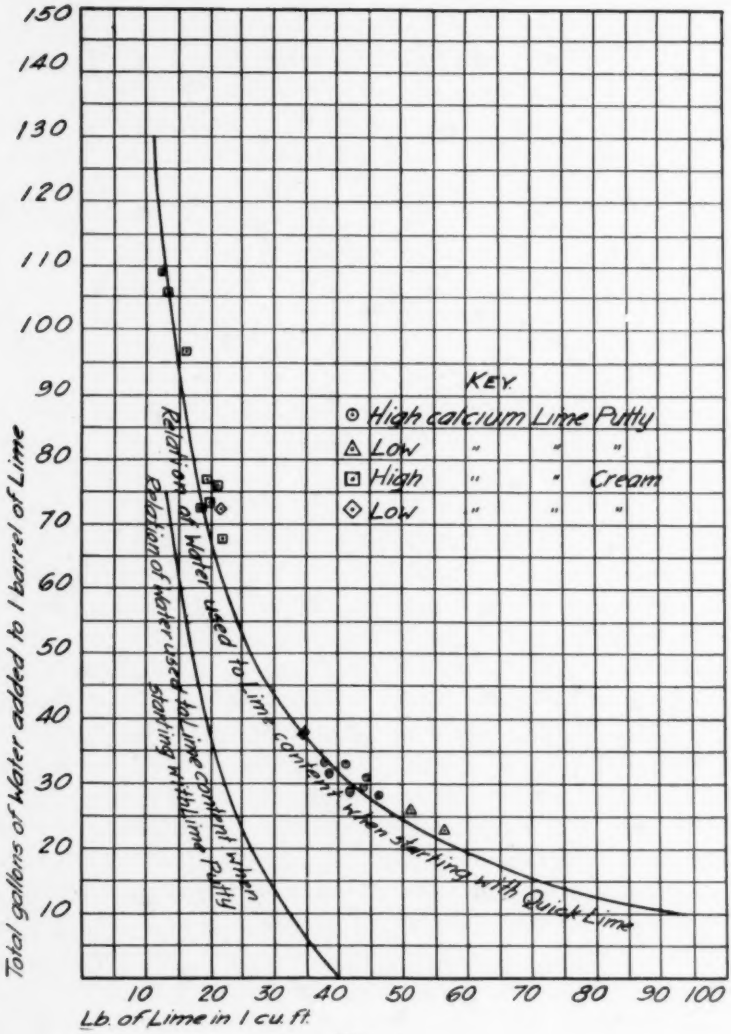


FIG. 1.

This curve is therefore of small practical value to the man on the job. Another curve is also plotted in Fig. 1 which shows the

relation between lb. of lime in 1 cu. ft. of mixture when the amount of water which is added to putty to reduce it to cream is known. This curve is based on the assumption that the average amount of water has been used to make the putty, which, from the average in Table I, is about 31 gals. per barrel of lime.

On this assumption the formula for the curve becomes:

$$Y = \frac{200}{\frac{x}{7.5} + 4.92}$$

In which all factors are the same as in the formula for the first curve except the constant 4.92, which represents the volume in cu. ft. of putty produced by 1 bbl. of lime.

By means of this curve, if it is known how many gallons of water are added to the putty produced by one barrel of lime to reduce it to cream, it can be determined how many pounds of lime are contained in each cubic foot of the cream, whatever its consistency.

Inasmuch as the amount of lime in a cubic foot of putty is a reasonably constant quantity, and as it is usual for lime to reach the stage of putty at some time during the process of preparation, it seems that this offers the best basis on which to build up a specification for mixing mortars.

If a high-calcium lime is to be measured when in the *putty state*, having a consistency of rather soft butter, it may with reasonable accuracy be assumed to contain 41.5 lb. of CaO per cu. ft.

If the lime is to be measured in the *liquid state*, having a consistency of good rich cream, in which condistion it is most easily mixed with sand and cement to a uniform mortar, then some means must be provided to determine the amount of lime in a given volume. The variation between different limes in the amount of water they require to make a cream of a given consistency is so great that no general figures can be safely applied.

These data are presented in a more usable and convenient form in Fig. 2 which shows the relation between the gallons of water added to 1 cu. ft. of putty of standard consistency to reduce it to cream and the number of pounds of lime in one gallon of the resulting cream regardless of its consistency.

The simplest way to measure lime putty is by the cubic foot, just as stone is measured: by using the bottomless box. The easiest way to measure water is by the gallon, either in a barrel or a 12-quart bucket. The easiest way to measure cream of lime is by the gallon or a 12-quart bucket. Using these most convenient units as a basis, from the curve given in Fig. 2 there can be determined quickly the number of pounds of lime in a gallon of cream if it is known how many gallons of water were used to thin each cubic foot of putty to produce the cream.

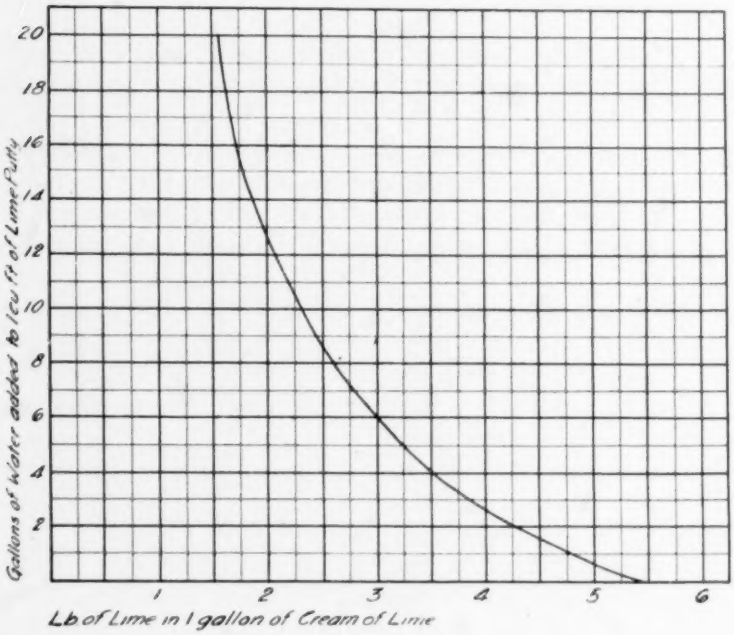


FIG. 2.

Suppose, for instance, that the plasterer prepared enough lime putty in advance to do a job of stucco. When ready to use it he reduced 5 cu. ft. of the putty to a cream by adding 50 gals. or one barrel of water or 10 gals. per cu. ft. The resulting cream, from the curve, would have 2.3 lb. of lime per gallon. This is found by following the horizontal line from the point in the left hand vertical scale representing 10 gals. per cu. ft. over to the

curve and from that point following the vertical line down to the lower horizontal scale which gives the result, namely 2.3 lb. per gallon.

Suppose that with this cream the plasterer wished to add 10 per cent of lime to a 2-bag batch of cement-sand mortar. This would mean that he would want enough of the cream of lime to give 19 lb. of CaO. As this cream contains 2.3 lb. per gallon he would have to add 8⅓ gals. or 2¾ buckets of 12 qts. or 3 gals. each.

It may be well to add here a caution against accepting the figures given above for general use until they have been either verified or modified by tests made on a much greater number of samples obtained from many parts of the country.

THE PROPERTIES OF PORTLAND CEMENT HAVING A HIGH MAGNESIA CONTENT.

By P. H. Bates.*

The attitude of the users of Portland cement in regard to the content of magnesia allowable in this material is rather difficult to understand, in view of the nature of the results upon which this attitude is based. In 1895 Dycerhoff presented a minority report of a committee to the German Portland Cement Manufacturers in which he gave results showing that a content of magnesia over 4 per cent tended to give poor cements and in a concrete made from cement with 8 per cent or more magnesia cracking was produced. This minority report has been used as a basis for many of the arguments against a high magnesia cement—the majority report of the committee which considered that magnesia up to 8 per cent was not harmful is seldom or never mentioned.

Since the above results were published there has been some investigative work† done on this subject, but invariably the results have been as contradictory as those published by the above-mentioned committee.

The following investigation, together with one not yet completed, was undertaken to throw some light upon the effect of magnesia not only upon the strength of the resulting cement, but also upon the other physical properties, the general appearance of the clinker and its condition in the kiln during burning, and upon the lime, silica and alumina compounds present in the ordinary cement.

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* Chemist, Bureau of Standards, Pittsburgh, Pa.

† Campbell, *Jour. Amer. Chem. Soc.*, Vol. 24, p. 969.
Michaelis, *German Port. Cement Mfrs. Assoc.* 1906.
Erdmenger, *Dingler Polytech. Jour.*, 252–135.
Jesser, *Zement Zeit.* 1–3–41.
Also any of the standard works on Portland cement, as *Meade's, Butler's, Candlot's, Spaldings.*

RAW MATERIALS USED AND THEIR BURNING.

The raw materials used were clay, limestone, dolomite and feldspar, having the following composition:

	Clay.	Limestone.	Dolomite.	Feldspar.
SiO_2	60.07	.08	.17	70.83
Al_2O_3	20.37	.12	.11	16.60
Fe_2O_3	7.23		.10	.19
CaO	.50	55.40	30.47	.44
MgO	1.85	.79	21.43	.08
Na_2O	.79	.....	.....	3.29
K_2O	2.51	.....	.....	8.32
Ig. loss	6.86	43.57	47.69	.63
	100.18	99.96	99.97	100.38

In the first burn, limestone, clay (and some feldspar to increase the silica somewhat without increasing the alumina to too great an extent) were used. In the successive burns, part of the limestone was replaced with dolomite to give increasing amounts of magnesia and decreasing amounts of lime until the next to the last burn was made when 18.98 per cent MgO was reached. The difficulties encountered in making these high magnesia burns and the resulting clinker were of such a nature that a burn using dolomite and no limestone was not made in this series, though in a latter series when the silica content was lower and the iron-alumina higher, such a burn was made.

After grinding the raw material together in a ball mill until 85 per cent passed a 200-mesh sieve, it was burned in the 2 x 20 ft. rotary gas-fired kiln used by this Bureau in its various cement investigations. It was soon found that the increase of magnesia produced quite a difference in the process of sintering within the kiln and in the appearance of the clinker. This was first noticed when the magnesia content had reached 9.50 per cent, when the clinker had a decided tendency to "ring up" in the kiln. The next increase in the magnesia content, which was to 14.07 per cent, caused a very marked change. Unless the clinker was produced in a slightly underburned condition, it started to dust within 24 hours after burning. Furthermore the increase in temperature necessary to pass from the slightly underburned condition to a hard vitreous state was very slight. The clinker produced there-

fore was the hard vitreous material which showed signs of dusting within 24 hours and is still (now almost 8 months old) dusting slowly. The color also was a reddish brown—decidedly different from the usual black, glistening, coke-like material produced when the magnesia was under 9.50 per cent. The ground cement has a very light reddish yellow tinge which differentiates it markedly from normal cement. The one burn made in this series which contained most magnesia (18.98 per cent.) produced clinker having these characteristics to a very marked degree. The second and third burns made were somewhat unsatisfactory—since, owing to low gas pressure, a satisfactory temperature could not be reached. Unfortunately also these three burns were those of low magnesia content; however, while the cement produced was not as volume constant as desired, yet, except in the second burn, the strength developed both as mortar and concrete was very good. The average temperature of each burn, readings having been made every 15 minutes with a Wanner pyrometer, was as follows:

	Deg. C.		Deg. C.
Burn 16	1520	Burn 29	1530
" 25	1430	" 30	1497
" 26	1486	" 31	1521
" 27	1512	" 32	1499
" 28	1505		

Without storing, the clinker was crushed in a jaw crusher, then passed through rolls until all passed a 20-mesh screen, and then introduced into a ball mill and ground with approximately 3 per cent plaster to the following finenesses:

	Passing 100 mesh.	Passing 200 mesh.
Burn 16	96.0	79.6
" 25	99.2	75.8
" 26	98.4	79.8
" 27	97.6	78.6
" 28	99.0	77.6
" 29	97.2	76.0
" 30	92.0	73.2
" 31	95.4	79.0
" 32	97.2	90.4

The chemical analyses of the resulting cements and their specific gravities are given in Table I.

PETROGRAPHIC ANALYSIS.

Of special importance is the petrographic analysis since but little work has been done in determining what minerals (or compounds) are found in clinker in which lime has been replaced by magnesia. Two very interesting papers have appeared comparatively recently on this subject and in both will be found resumés of previous investigators.* Neither of these, however, attempted to correlate the results with the physical properties of their burns. However, Klein and Phillips working in this laboratory found that magnesia could occur in Portland cement up to 7.5 per cent before it would appear as a new compound not ordinarily present—that is, below this percentage it would form homo-

TABLE I.—CHEMICAL ANALYSES AND SPECIFIC GRAVITY OF CEMENTS BURNED.

Burn No.	16	25	26	27	28	29	30	31	32
SiO_2	21.49	21.81	22.08	22.08	22.09	22.61	22.71	23.12	22.84
Al_2O_3	6.77	7.08	7.60	7.51	7.11	7.93	7.07	7.67	7.78
Fe_2O_3	2.88	2.57	2.69	2.94	2.77	2.82	2.96	3.31	3.41
CaO	64.54	62.68	61.37	60.16	59.60	56.88	55.54	48.00	44.09
MgO	1.77	3.27	4.20	5.12	6.49	7.41	9.50	14.07	18.98
SO_3	1.62	1.43	1.54	1.65	1.10	1.60	1.63	1.58	1.61
Na_2O	.07	.10	.10	.12	.14	.09	.15	.18	.28
K2O	.34	.57	.32	.26	.30	.34	.34	.41	.52
Ig	.52	.71	.38	.36	.61	.64	.46	1.35	.60
Total	100.00	100.22	100.28	100.20	100.21	100.32	100.36	99.69	100.11
Specific Gravity	3.192	3.157	3.200	3.234	3.216	3.259	3.243	3.225	3.178

geneous compounds with the lime in either the β orthosilicate or 3—1 aluminate.

In this investigation no new compounds were found until Burn 30 was reached (9.5 per cent MgO) when Monticellite—$MgO.CaO.SiO_2$—was noticed in very small quantities. Burn 31 showed a much larger amount of this compound and in addition Spinel—$MgO.Al_2O_3$—was noticed in a small amount. Burn 32 contained these in still larger amounts. It should be noted also that the Monticellite is colored by iron oxide, whereas the Spinel is not.

Both of these compounds are found in nature and are usually considered to weather very slowly—they therefore do not hydrate (excepting possibly after very long periods). Further they are

* Jesser. *Ton Zeitung*, Oct., 1911, page 1476.
Klein and Phillips. *8th Int. Cong. Appl. Chemistry*, Vol. 5, page 73.

considered insoluble in dilute and concentrated hydrochloric acid, and it was found that these high magnesis cements contained a considerable amount of insoluble residue—in fact, in making the analyses, the concrete was first fused with sodim carbonate.

The Table II is given an approximate quantitative determination of the constituents present in these burns. These were made in the same manner as described in the paper* read before the last meeting of this Institute. It is seen in this table that the orthosilicate has been very materially decreased in the latter burns of this series (Burns 25 and 26 are really abnormal owing to their underburning, which produces a low amount of trisilicate and a large

TABLE II.—PETROGRAPHIC ANALYSES OF THE VARIOUS BURNS.

	Free Lime.	Trisilicate Trialuminate.	β orthosilicate.	Ferrite.	Monticellite.	Spinel.
Burn 16	.2	61.6	30.6	7.6	None	None
25	3.7	38.9	47.0	10.4	"	"
26	2.4	43.2	45.3	9.1	"	"
27	.4	51.8	39.6	8.2	"	"
28	1.5	47.9	41.6	9.0	"	"
29	Trace	49.8	40.3	9.9	"	"
30	None	54.1	35.7	10.2	†Present	"
31	None	56.0	32.4*	8.3	2.2	‡Present
32	None	57.1	20.7	7.8	11.5	‡Present

* β' orthosilicate also present (1.1 per cent).

† Present in such small quantities and so poorly developed as not to be distinguishable from the β orthosilicate.

‡ Owing to its optical properties Spinel cannot be readily distinguished from calcium trialuminate in thin sections, in this respect being like the trisilicate—hence this constituent appears with these two others.

amount of orthosilicate). With this decrease of orthosilicate there has been a very marked increase of the trisilicate, since the amounts given in this heading include also the Spinel, which appears only in the higher magnesia burns. It would appear therefore that the Monticellite has taken its silica more from the ortho than the trisilicate.

The sections prepared from the clinker of these burns, show the trisilicate very well developed—particularly in large needle or columnar aggregates. While this has been noticed before in clinker of normal magnesia content, it is invariably present in this condition in the clinker of high magnesia content. See Microphotographs, Fig. 1 and 2.

* See Proceedings, Vol. IX, 1912.—Ed.

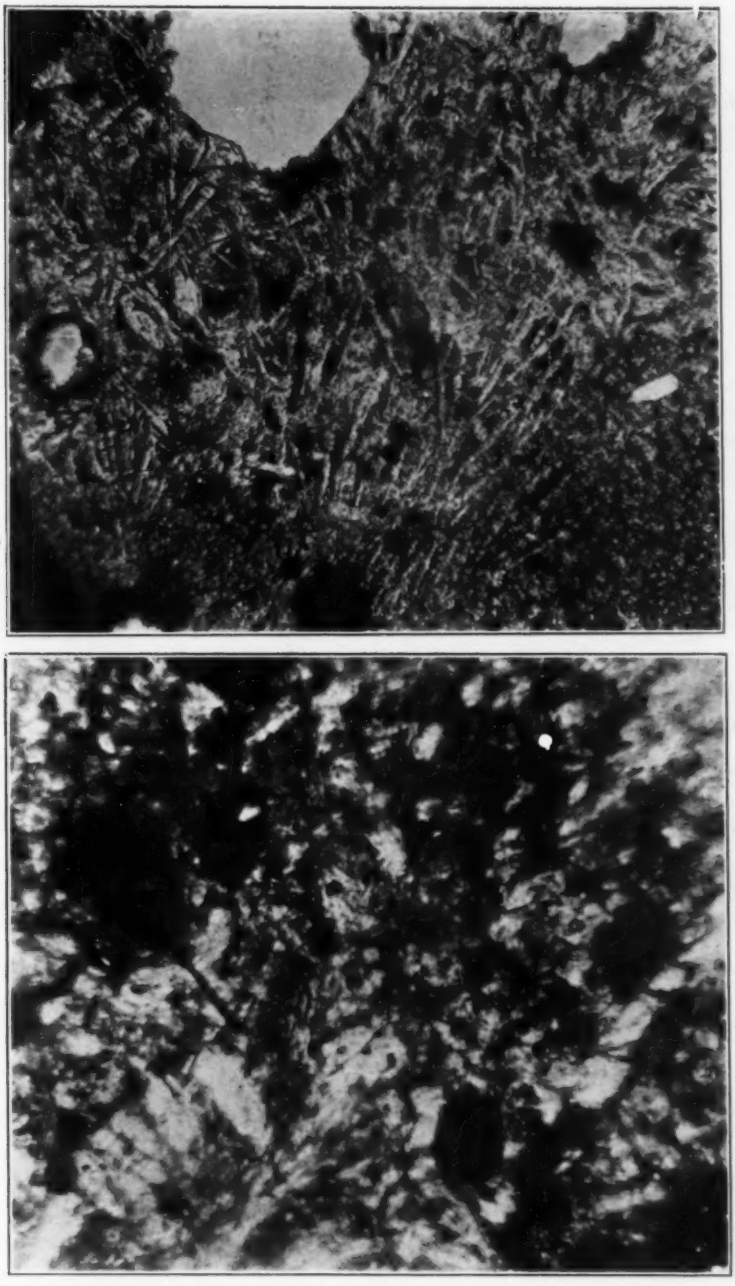


FIG. 1.—MICROPHOTOGRAPHS OF THIN SECTION OF 2 PIECES OF CLINKER SHOWING THE NEEDLE DEVELOPMENT OF $3CaOSiO_2$. MAGNIFICATION = 165×.

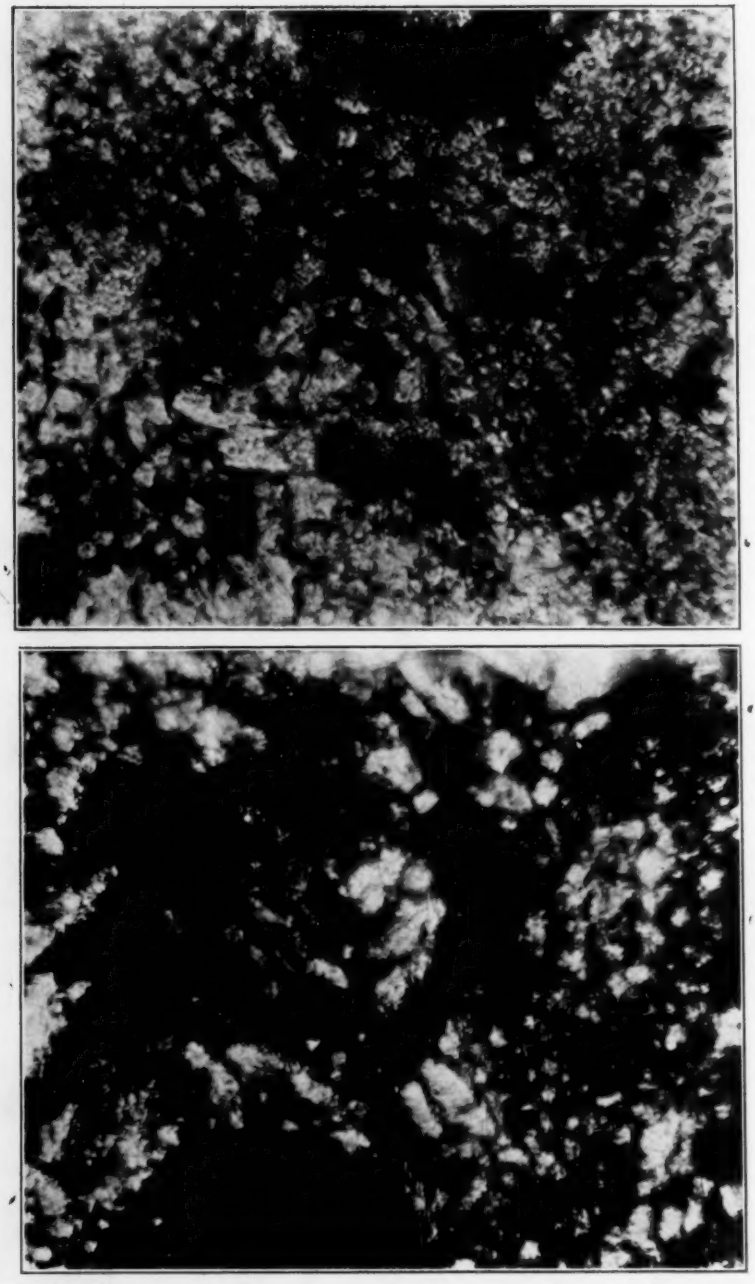


FIG. 2.—MICROPHOTOGRAPHS OF A THIN SECTION OF HIGH MgO CLINKER, (A) WITH NICOLS OUT AND (B) WITH CROSSED NICOL. THE WELL DEFINED MATERIAL WITH DISTINCT CRYSTALLINE OUTLINE IS MONTICELLITE. ALL MATERIAL WHICH IS DARK IN (B) BUT WHITE OR GRAY IN (A) IS LARGELY ORTHOSILICATE OF LIME. MAGNIFICATION = 375×.

TIME OF SET AND CONSTANCY OF VOLUME.

In Table III is shown the initial and final sets (determined both by the Vicat and Gilmore needles), the normal consistency and the soundness of the 28-day air and water pats as well as the steam and boil pats.

From this table it is seen that so far as soundness is concerned, Burns 25 and 26 were unsatisfactory,—the former sufficiently so to cause lower strengths than the preceding or succeeding burns—the latter, however, produced some of the highest strengths obtained. Burn 32 had not obtained a set at the end of 24 hours, consequently the pats tested then, failed.

TABLE III.—TIME OF SET, NORMAL CONSISTENCY AND CONSTANCY OF VOLUME.

Burn No.	Normal Consistency.	Time of Set. Gilmore. Initial hrs. min.	Gilmore. Final hrs. min.	Vicat. Initial hrs. min.	Vicat. Final hrs. min.	Pats. Boiling.	Steam.	28 day air.	28 day water.
16	20.0	3 : 00	8 : 05	: 55	6 : 40	OK	OK	OK	OK
25	21.0	3 : 05	8 : 30	2 : 15	6 : 10	Disintegrated		OK	OK
26	20.5	3 : 45	9 : 15	2 : 15	7 : 30	"	"	OK	OK
27	19.0	4 : 00	8 : 30	2 : 40	6 : 15	OK	OK	OK	OK
28	21.0	2 : 35	6 : 30	1 : 35	5 : 10	OK	OK	OK	OK
29	20.5	4 : 25	9 : 00	1 : 20	6 : 30	OK	OK	OK	OK
30	19.0	3 : 20	13 : 00	1 : 00	9: 30	OK	OK	OK	OK
31	18.0	: 06	2 : 05	1 : 05	1: 25	OK	OK	OK	OK
32	31.0	5 : 00	48 : 00	1 : 00	30 : 00	Soft and cracked		OK	OK

The effect of the increasing magnesia upon the set was not noticeable until Burn 30 (9.50 per cent MgO), when a rather quick initial set was developed, but the final set was quite slow. The next burn (31, containing 14.07 per cent MgO) developed a very rapid initial and final set with the evolution of much heat. The burn containing the most magnesia (32, containing 18.98 per cent) developed a very slow initial set with the Gilmore needle, though with the Vicat it was rather quick. This was due to a rather marked evolution of heat. The final set of this burn was very slow. It is evident that not a sufficient number of burns was made with a magnesia content lying between 9.50 and 16.98 per cent upon which to base any law in regard to the effect of this constituent upon the set—thus it could not be said that as the magnesia increased from 9.50 to 14.0 per cent there would be a

material with constantly increasing quick set and that as the percentage is increased beyond 14 per cent it again becomes slower.

These higher magnesia burns develop considerable heat at about the time of initial set and thereafter there is no real final set of the character obtained with cement of normal magnesia content—but it seems to dry out and leave a mass bonded together to only a sight degree. The burn containing 14 per cent magnesia evolved so much heat that this drying out occurred comparatively early, giving what is reported as final set at an early period, whereas the highest magnesia burn did not give such a high rise and the final set was at a much later period. No explanation can be given in regard to this evolution of heat, as none of these burns contained either free lime or magnesia—the constituents which usually cause such phenomena, and furthermore the aluminates also were of the same composition and of about the same amount.

STRENGTHS DEVELOPED.

Tensile briquettes of both neat and 1 : 3 standard Ottawa sand, and 2 in. neat and 1 : 3 standard sand cubes for compression, were made for periods extending over a number of years. These are being stored in water. In addition to these, 6-in. cubes of 1 : 1 $\frac{1}{2}$: 4$\frac{1}{2}$ gravel concrete were also made, half of them are being stored alternately in water and air and the other half in a 2 per cent solution composed of equal parts sodium chloride, magnesium chloride, sodium sulphate and magnesium sulphate, for certain periods and then removed and allowed to dry out in the air. Thus the test pieces broken at the end of 28 days had been in the water (or solution) and air during alternate weeks; "90-day test pieces" were in the water (or solution) for 5 one-week periods and 1 two-week period with intermediate periods of drying of one week each; the "180-day test pieces" were stored in the water (or solution) for the same periods as the "90-day test pieces" and in addition for 2 three-week and 1 four-week periods, with intermediate periods of one week drying. All test pieces after this period of breaking will be immersed for four-week periods and then dried for two-week periods.

The storing of the test pieces in this solution is carried out in order to see if the salts present will materially affect the concrete.

As is well known, there is a belief that cement for use in sea water should have less magnesia than that for use elsewhere. As quantities of sea water were not available this solution was used, which contains some of the salts found in sea water and in addition has the advantage over the latter in that it contains the supposedly most deleterious salts—the magnesium salts—in greater quantities.

Previous work in this laboratory as well as observations of concrete work exposed to sea or alkali water, has shown that alternate wetting and drying produce the most rapid disintegration, conse-

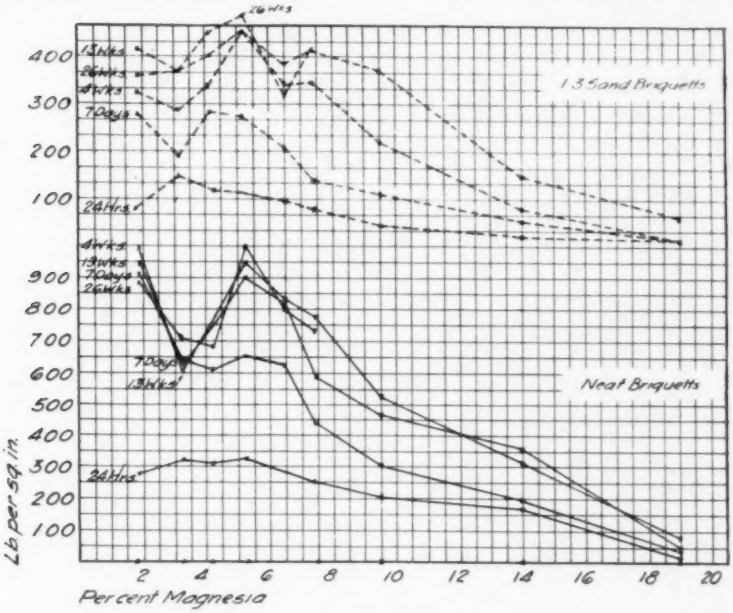


FIG. 3.—STRENGTH OF NEAT AND 1 : 3 SAND BRIQUETTES.

quently this method of storage was adopted. Furthermore, as the test pieces thus treated were to be compared with others stored in water alone, the latter were also subjected to the same periods of drying; in other words these were treated as the former, only water instead of the solution was used. In all cases the test pieces were removed from the molds at the end of 24 hours, and immediately placed in the tanks of water (or solution) and all were tested after the end of one of the drying periods. Burn 32 had developed so little strength in the small test pieces

TABLE IV.—STRENGTH IN LB. PER SQ. IN. OF NEAT, SAND AND CONCRETE TEST PIECES.

Burn No.	Kind of test piece.	Tension.					Compression.				
		24 hours.	7 days.	4 weeks.	13 weeks.	26 weeks.	24 hours.	7 days.	4 weeks.	13 weeks.	26 weeks.
16	Neat	275	925	965	951	893	2528	11343	13469	14238	14849
	1:3 sand	72	291	322	410	365	398	2039	3516	4522	4199
	1 : 1½ : 4½ concrete, Stored in water	...	...	...	...	...	....	.....	3491	4067	4081
	1 : 1½ : 4½ concrete, Stored in solution	...	...	...	...	...	....	.....	3883	3829	3435
25	Neat	329	645	623	608	704	2955	5232	6840	7610	9162
	1 : 3 sand	156	193	281	363	362	712	1284	1999	2656	3204
	1 : 1½ : 4½ concrete, Stored in water	...	...	...	...	...	....	.....	2910	3625	3765
	1 : 1½ : 4½ concrete, Stored in solution	...	...	...	...	...	....	.....	2932	3756	3951
26	Neat	305	609	756	764	688	3102	5818	9851	10855	11710
	1 : 3 sand	128	291	336	403	445	555	1461	2725	3353	3761
	1 : 1½ : 4½ concrete, Stored in water	...	...	...	...	...	....	.....	3275	3776	4579
	1 : 1½ : 4½ concrete, Stored in solution	...	...	...	...	...	....	.....	3351	4261	4564
27	Neat	329	657	911	957	1015	3015	7614	10812	11821	15388
	1 : 3 sand	108	275	445	443	580	515	1326	3042	3379	4754
	1 : 1½ : 4½ concrete, Storedinwater	...	...	...	...	...	....	.....	3410	4228	4605
	1 : 1½ : 4½ concrete, Stored in solution	...	...	...	...	...	....	.....	3427	4194	4318
28	Neat	292	627	821	827	812	2833	5560	7206	10363	13181
	1 : 3 sand	99	212	339	381	321	640	1380	2504	3548	3715
	1 : 1½ : 4½ concrete, Stored in water	...	...	...	...	...	....	.....	2484	3493	4185
	1 : 1½ : 4½ concrete, Stored in solution	...	...	...	...	...	....	.....	2466	3173	3771
29	Neat	258	445	591	775	723	2268	4184	6787	9438	10923
	1 : 3 sand	91	149	341	427	430	395	879	1708	2618	3368
	1 : 1½ : 4½ concrete, Stored in water	...	...	...	...	...	....	.....	2059	3171	3431
	1 : 1½ : 4½ concrete, Stored in solution	...	...	...	...	...	....	.....	1675	2813	3119
30	Neat	209	300	464	543	...	1318	3078	4794	8112	.....
	1 : 3 sand	58	110	217	378	...	227	454	1324	1809	.....
	1 : 1½ : 4½ concrete, Stored in water	...	...	...	...	...	....	.....	1616	2254	2589
	1 : 1½ : 4½ concrete, Stored in solution	...	...	...	...	...	....	.....	1305	2343	2563
31	Neat	170	196	359	311	...	Not made		.....	.....	.....
	1 : 3 sand	29	52	79	160	...	....	.....	Not made		.....
	1 : 1½ : 4½ cencrote, Stored in water	...	...	...	...	...	....	.....	483	1040	1248
	1 : 1½ : 4½ cencrote, Stored in solution	...	...	...	...	...	....	.....	533	1342	1488
32	Neat	27	30	38	77	...	292	330	322	564	.....
	1 : 3 sand	24	26	27	68	....	117	133	140	288	.....
	1 : 1½ : 4½ concrete, Stored in water	...	...	...	...	...	....	.....	....	.....	.....
	1 : 1½ : 4½ concrete, Stored in solution	...	...	...	...	...	....	.....	Not made		.....

that no concrete was made from it. Burn 31 moreover was not removed from the molds until the end of 48 hours.

The strengths of these various test pieces are shown in Table IV and Fig. 3, 4 and 5. While the small tensile test pieces show many of the abnormalities of tensile test pieces, yet they follow to a marked degree the general trend of the strengths developed

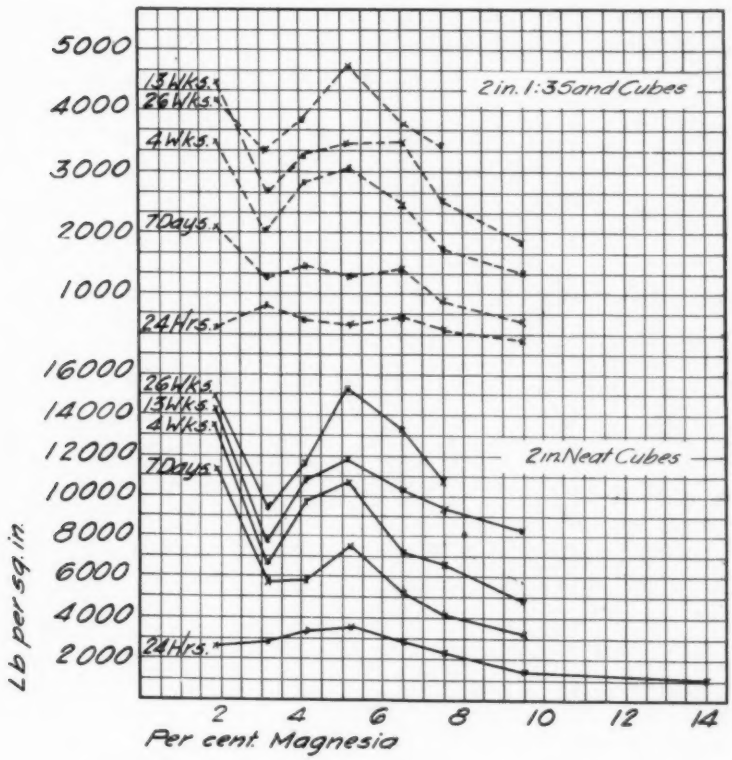


FIG. 4.—STRENGTH OF 2-IN. CUBES, NEAT AND 1 : 3 SAND.

by both the small and large cubes. Thus in every case it is seen that Burn 25 shows the effect of underburning which was noticed in the soundness tests, and further that the maximum strengths were developed in Burn 27, containing approximately 5 per cent MgO. It should be stated that the actual burning of this cement was more satisfactory than any of those containing less magnesia,

—the proper temperature for burning seemed to be a little lower than that required for the former burns and further, the clinker had a much better appearance. The latter burns containing more magnesia, showed to even a more marked degree, the lowering of the clinkering temperature with increasing magnesia,—but the strengths developed are also shown to be less. However, Burn 29, containing approximately 7.50 per cent magnesia, still has satisfactory strengths, giving for instance very excellent sand briquettes at 28 days and longer periods.

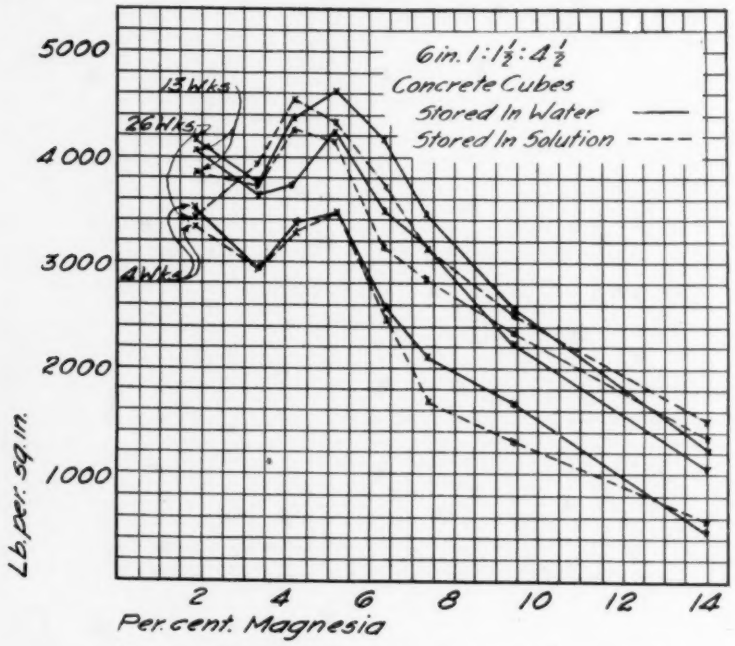


FIG. 5.—STRENGTH OF 6-IN. CUBES, 1 : 1½ : 4½ CONCRETE.

It can also be seen that these high magnesia cements (those in which the magnesia is not high enough to form new compounds) gain their strength somewhat slowly and give better sand test pieces than neat ones—apparently the aluminates and silicates in which part of the lime has been replaced by the magnesia, hydrate more slowly.

Those cements in which the magnesia has formed the new compounds, show such low strengths as to make them unsuitable

for use as a cementitious material. Thus Burn 30, which just shows the appearance of these compounds, at early periods gave quite low values, though in the sand test pieces at later periods it has more than passable strengths, but still more magnesia makes the cement entirely unusable.

The results obtained for the strengths of the concrete cubes show conclusively that so far the salts used in the solution have no deleterious effect—in fact, if one were to take the results obtained with the cement containing the highest magnesia (14.07 per cent) from which concrete was made, the conclusion would be that the concrete was improved.

The magnesia present in cement can only affect the mortar or concrete made from it by producing lower strengths due to slower hydration (or lack of hydration) as a result of a change produced in the constituents normally present in the cement or the actual production of new constituents—it cannot affect the strength by reacting with magnesia salts in solution such as are present in sea or alkali water. It is not the magnesia of cement which reacts with the magnesia salts of the water, but the lime freed during hydration. Therefore the addition of a silicious material (puzzuolana, tufa, etc.) to concrete which may combine with this freed lime has at least a theoretical basis, since the resulting lime silicate will have removed this lime from the sphere of reaction with the magnesia salts and consequent precipitation of hydrated magnesia and rather insoluble lime sulphate compounds. It might even be argued that since the high magnesia cements contain less lime, they would consequently be better for sea water concrete, the magnesia of the cement being non-reactive with the salts present in the water. Such would be the actual fact, but as shown, above 9.5 per cent MgO there are constituents present in the cement which are non-hydraulic and consequently being inert reduce the strength so materially as to produce an undesirable product. But from the results so far obtained, a cement containing less than this amount of magnesia would be just as satisfactory for any one use as another.

SUMMARY.

1. Cement can be readily burned in a rotary kiln with a magnesia content of about 9.50 per cent before the resulting

clinker will be materially different from that of a cement containing 3 per cent or less. Very high magnesia clinker (over 14 per cent) is very vitreous and dusts slowly, starting within about 24 hours.

2. Petrographically no new constituents are present until about 9 per cent MgO is reached, when small quantities of Monticellite are noticed, with still increasing amounts Spinel occurs. Both of these are non-hydraulic and if the cement is treated with dilute or concentrated hydrochloric acid, are largely insoluble.

3. Pats made of a cement as high in magnesia as 18.98 per cent were sound after 28 days in air and in water, when placed in boiling water or steam at the end of 24 hours they cracked slightly but were no softer than when removed from the molds. Pats made from cement with lower percentages of magnesia remained in excellent condition after all tests.

4. Cements containing up to 9.6 per cent magnesia showed normal initial set with a slight tendency toward slower final set. With a greater content, there was a quick initial set accompanied by a very marked rise of the temperature; with the highest magnesia cement made, the initial set determined with the Vicat needle was quick, accompanied by a rather marked evolution of heat,—This heat being rapidly dissipated in the smaller, less insulated pat gave a slow initial set with the Gilmore needle. The final set was very slow.

5. The strength developed either by the neat cement or 1 : 3 mortars or 1 : 5 concrete up to periods of half a year, show that cements containing as much as 7.5 per cent magnesia are satisfactory. Further, the increasing magnesia reduces the rate of hardening, so that at the late periods cements with a higher content have rather commendable strengths. Up to this period a 2 per cent solution of equal parts sodium chloride, sodium sulphate, magnesium chloride and magnesium sulphate has no injurious effect.

Acknowledgments are due to Mr. A. A. Klein for the petrographic examinations and to Mr. E. R. Gates for superintending the molding and breaking of all test pieces.

THE LAYOUT OF CONCRETE PRODUCTS PLANTS.

By E. S. Hanson.*

It is a matter for congratulation that the manufacture of concrete products has developed beyond the "back yard" stage, and up to the point where it is pretty generally recognized that a specially designed plant is necessary if one expects to turn out these products with the greatest efficiency. The pioneer who thought a shed in the back yard a sufficient start in this business, is, however, not to be too severely condemned; for in the first place he in most cases knew no better, and in the second place he was entering an unknown field and did not care to risk more than was absolutely necessary.

Sometimes even this shed was dispensed with. The writer has in his files, and cherishes it as a most interesting historical record, a photograph showing concrete blocks being made, cured and laid up in the wall, almost in one operation, and not so much as a piece of canvas to keep off the hot August sun. The blocks probably received a fair curing if it happened to rain several days in succession; otherwise they had to go through with such sprinkling as they happened to get when someone had nothing else to do. Is it any wonder that they were but little removed from the sun-dried mud bricks of savage tribes, and that they served to bring the concrete block into disrepute in that particular locality? This manufacturer considered that he was practicing a clever scheme by taking his equipment around from one job to another and making the blocks right on the spot. He saved hauling the blocks, for one thing; and then it was so convenient to make special shapes as they were needed; these being handed almost direct from the machine to the masons who were laying up the wall.

This, happily, is a thing of the past; and yet many manufacturers still have a long way to go before they attain that perfection of product or efficiency of manufacture which the industry is ere long going to demand of them.

* Editor, *The Cement Era*, Chicago, Ill.

Concrete products are now for the most part housed either in plants built especially for the purpose, or in old buildings which have been converted permanently for this purpose by considerable remodeling, but it is certainly not too much to state that the limit of efficiency in plant layout and equipment has not as yet been attained. There is, however, no reason why this development should not now go forward quite rapidly; for the concrete product—be it block, tile, post, structural member, ornamental piece, or what not—is no longer an experiment, and a man of fair business acumen, who uses judgment in locating himself with reference to his markets and his raw materials, need not hesitate to invest in such a plant.

All the local conditions will, however, need to be gone over thoroughly in order to determine the market probabilities. In the case of a plant for the manufacture of drain tile, for instance, this will include such considerations as the nature of the soil, whether adapted to improvement by tile drainage or not; the value of the land; the general prosperity of the people, and their attitude with respect to improvements in farming, and their attitude with reference to spending money; the feeling in the community, and especially among county or drainage district officers, with reference to cement products. All these things will not only influence one in determining a point for the establishing of a plant, but when such location is finally determined upon they will help the owner to decide what size the plant shall be and what investment he will be warranted in making.

A temporary plant for the accomplishment of a particular piece of work can of course be located almost anywhere, the proximity to the work sometimes offsetting heavy delivery charges for materials; but for an ordinary commercial plant, which expects to continue in business year after year, it is necessary that the item of haûling be cut down as much as possible. The writer has seen plants which were apparently located on no other consideration than because of proximity to the owner's home, or because of the securing of a site at a very low figure. Materials had to be secured at some distance by team haul, and the finished product likewise had to be hauled too far to the railway station for rail shipments, while the location was not such as to attract attention and advertise the business.

Of course it sometimes happens that an excellent market is discovered in a locality where there are no materials of a suitable nature. In cases of this kind it is usually more economical to provide for delivery by rail, rather than to try to locate a source of supply within teaming distance. The sidetrack can then be used for the delivery of cement as well as for the shipment of the finished product. In fact, there are few plants now being opened up which do not have direct connection with the railroad, and the design of the plant is so made that the materials of all kinds, as well as fuel, can be delivered from the track direct to the storage bins.

In general it might be said that in this as in other lines of work it is usually best to work along the lines of least resistance. It is scarcely worth while to establish a plant where the sentiment is strongly adverse to cement, when there are so many good locations not yet occupied. The missionary work which would have to be done in such a location before one could get a business foothold, will be done sooner or later in the general spread of the knowledge of the advantages of cement, without tying up anyone's business investment in the meantime.

It would of course be the logical thing for the buildings of a plant to be constructed of its own product; but a plant is scarcely able to produce structural material until it is housed and in working order. The manufacturer is thus placed in the position of the Irishman who "couldn't get his boots on until he had worn them for a time or two." Sometimes, however, the equipment can be set up temporarily and sufficient units made to construct at least the most needful part of the housing; but it must be pointed out that such units are likely to be below the standard of excellence which will later be attained, and will prove a poor advertisement for the business. The practice cannot be too severely condemned of using cull blocks for a manufacturer's own buildings, except it be for foundations, partition walls, or other places where they will not be seen by the public. Sometimes, too, a miscellaneous lot of blocks is used for such buildings, of various faces, types and colors, thrown together as they happen to come to hand. This is all wrong. A slight saving is effected this way, but the loss in prestige is incalculable. A plant should be, to a certain extent, its own advertisement, especially in view of the fact that it is

likely to be located near the railroad, where it will be the subject of observation from passing trains. But whatever its construction, it should be of a good grade, built with a view to permanence, service, and a certain measure of sightliness.

The kilns should be of concrete by all means. Then they are built for all time. If the plant is manufacturing blocks, the kilns can easily be constructed one at a time after the main part of the plant is up and the machines in place, using blocks for the walls and a simple form of reinforced concrete arch for the roof. This roof can be built up over a rough wood form, or can be constructed without centering on some one of the patented types of metal designed for this purpose. In this latter case, however, it should be carefully back-plastered in order to preserve the metal from the corroding effect of the steam. The curing room of building paper is being less frequently met with, and will ultimately be crowded out entirely by the demand for permanent construction.

No plant, in these days of the rapid development in the use of concrete, should be built without a consideration of future possibilities as well as present needs. The manufacturer who has the facilities for installing the new device which may be invented tomorrow, is the one who will have first opportunity to reap the benefit of its advantages. A few additional feet of floor space for which there seems to be no immediate need can well be afforded, and costs little more if included in the original construction. This space, it need scarcely be stated, should be in length of work room rather than depth; that is, it should be in the direction of the main transfer track, so that ready access could be had to this track for any future installation.

Curing chambers are separate units and can be added on from time to time as needed, provided only that the plant is so designed beforehand as to make this possible. For instance, a street on one side and a railroad siding on the other might prove an effectual bar to the construction of more curing space without an entire reconstruction of the plant. This is a contingency which should not be allowed to happen.

It is quite important that the doors of kilns be so constructed that they may not be in the way when open. An outward swinging door is the worst imaginable, though it is still to be found in

a few plants. It not only is likely to become a frequent obstruction, but often is so loosely made that it sags on its hinges and will not close the kilns tightly. A sliding door has either to cover adjoining kilns or allow of opening only one half the chamber which is in service. A door which is removable and held in place by a cleat resting in sockets, gives good service but is unnecessarily clumsy to handle.

Of the various types of rigid doors, perhaps the upward swinging door is the best for the inner end of the kilns. Such a door can be held up by a simple latch, is entirely out of the way, is easy to handle and its weight keeps it closed when down. At the outer end of the kiln an upward sliding door can be used to advantage. In some cases this is counterbalanced with a weight, making it very easy to operate.

The canvas curtain gives fairly good service if it is properly taken care of, but it is too often allowed to flap almost uselessly, and it is short-lived at best. A device was put upon the market some little time ago for handling a curtain, combining a roller and guides, somewhat after the fashion of the curtain to a Pullman car window; but the writer has never seen it actually installed. Canvas curtains can be made practically steam tight by having stationary strips nailed on to the walls at the sides of the curtains and other strips hinged to these with screen-door hinges. When the curing room is filled, the curtain is lowered and these movable hinged strips are turned down over the edges of the curtain, the spring hinges holding them tightly in place and making a practically steam-tight joint. In addition to this precaution, the canvas curtains may be painted so that no steam can penetrate through them.

A concrete floor throughout the plant will be found well worth while. In addition to its durability, its rigidity and smooth surface commend it for the efficient moving of products, while it serves well in many cases as a molding floor, or as a curing floor for setting out sewer pipe, large tile, etc.

The great consideration in the design of any such plant is the elimination of needless work, the placing of men, machines and materials in such a way that work will be accomplished with the least possible effort.

Some of the points already mentioned, and a number of

others, are brought out in the description of some typical plants given below. None of these are shown as being perfect types, and some of them are admittedly for the purpose of calling attention to inefficiency in design. It is hoped that they will, however, serve to stimulate an interest in this matter and by so doing assist in the development of more nearly perfect plants for the manufacture of concrete products.

Fig. 1 shows one of the very first cement tile plants ever established and is shown not because it has any characteristics worthy of duplication, but simply to illustrate the advance which

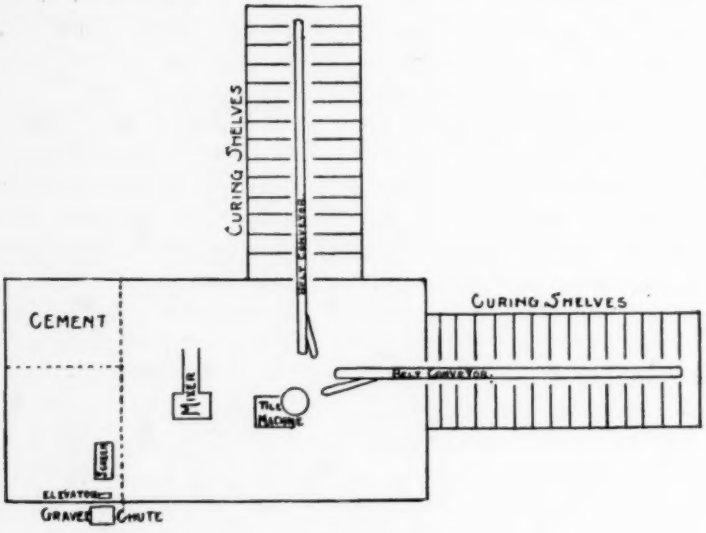


FIG. 1.

the industry has made. So far as the writer knows, this is the only plant ever equipped with a belt conveyor for delivery. This was before the car and track system had come into use in very many plants, and was, in fact, at the time when the industry was in a more or less experimental stage and when plants were being established and equipped on just as small an investment as possible. These belt conveyors were installed at a very small expense and were possibly an improvement over having men run back and forth with the tile. The jackets containing the tile were placed on the upper belt and carried along until taken off by men

stationed along the belt in the alleys. The empty jackets were then placed on the return belt down below and sent back to the main work-room where a shunt was provided to remove them from the belt. The principal objection to this system was found to be that the tile were jarred in passing over the rollers or bumping against the sides of the carrier, and too large a percentage of them went down. This was obviated to a certain extent by placing a strip of steel on the rollers under the belt. The system had little to recommend it, however, even with this modification, and it has not been duplicated. It is understood that a new type of conveyor, which has none of the drawbacks of such a one as here shown, is just being perfected by a well known manufacturer.

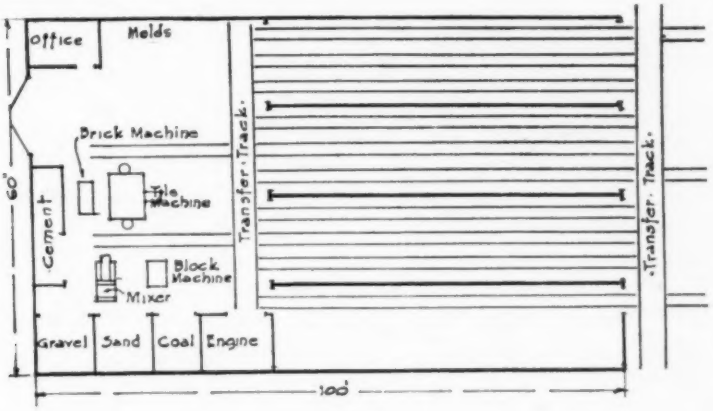


FIG. 2.

This plant is also of the obsolescent type because of the rack and shelf method of curing. This method was originally in use in practically all concrete products plants because it was easy to install, it cost little, and if the business did not prove a success it was therefore little loss. Then, too, the great advantages of steam curing had not become generally known, not only because of its uniform curing of the product, but also because of getting tile more rapidly through the plant, with less handling and with greater economy in every way.

This plant, Fig. 2, is shown principally to emphasize the fact that the kilns are entirely too large. This space should have been

divided into four or five kilns at least. Too many concrete products are ruined at the outset by leaving them in a drafty curing room for the greater part of a day while sufficient units are being made to fill the chamber. This is all wrong. The curing rooms should be of such size that they can be filled and closed up at least twice a day, that is, one curing room should not be larger than to accommodate half a day's run. A size which has proven very efficient is 4 ft. wide to allow one track 6 ft. high to allow workmen to enter conveniently, and long enough to accommodate either three or four cars.

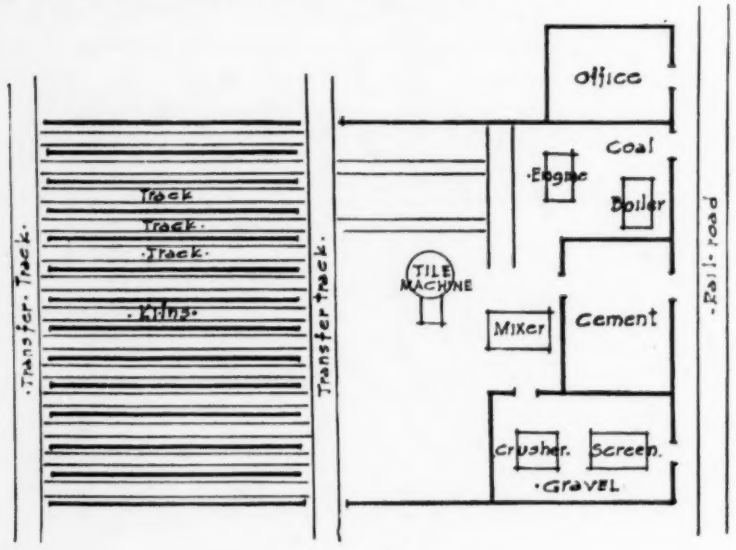


FIG. 3.

A conveniently arranged plant and rather simple in construction is shown in Fig. 3. Along the railroad track are the rooms for gravel, cement and coal, also the office, so that deliveries of materials can be watched and checked up. There is no arrangement for delivery of materials to the mixer, other than by hand, but the mixer is conveniently located to both gravel and cement. The kilns would seem to be of just about the correct size, with one track each, accommodating three or four cars.

We might comment right here on the advantages of car and track system, although this method of handling is coming into

such general use as to be its own recommendation. The car is in effect a traveling storeroom. With a track conveniently placed to the machine which the car is serving, the product can be delivered by scarcely more than a movement of the body. The loaded car, containing, possibly, anywhere from 20 to 300 units of product, is then taken to the curing rooms in but little more time than a man would require to carry a single unit under the old shelf and rack system. This saving in time is also duplicated when it comes to removing the product from the curing room, as it can be immediately transferred to the yard without handling, except when it is taken from the car to the stock pile.

The layout of a track system should be well planned before being installed, however, so as to insure the greatest efficiency. In plants where the product is continuous and uniform in type, the tracks on the working floor can be made stationary and sunk to the floor level, but in plants which vary considerably in the nature of their work, a system of movable tracks can sometimes be used to advantage. In such plants, however, screw anchors should be placed in the floor, so that the tracks can be securely fastened in place.

A system of cars and tracks runs up into money quite rapidly, so that many plants make the mistake of having insufficient equipment. Nothing is gained in the long run by limiting the number of cars below what can be used to advantage, while a few additional feet of track will often pay for itself in a short time by decreasing the amount of carrying.

Fig. 4 shows a plant in a building about 60 ft. long and 40 ft. wide. There are four departments in the plant, each having its own space definitely marked out and arranged for best results in operation: the power plant, material storage, manufacturing, and curing.

At the end of the building are the bins for the storage of materials. There are five of these, made up of four concrete partitions about 9 ft. high, extending out 10 or 12 ft. from the end wall of the building. In these bins are kept the different sizes and grades of sand, and the cement and color materials in immediate use. The bin walls were built with a view to placing above them a revolving screen to receive bank or pit sand from an elevated platform outside of the building, and deliver it

screened to the various bins. The storage bins are on a 4-in. concrete floor, extending out over the adjacent section of the building, which is devoted to the making of the blocks.

There are three block machines and a brick machine in the plant equipment. These are set in line across the building in front of the bins. Space is provided for two lines of four machines each, a total of eight machines. Strips of wood are laid in the concrete floors, to which the machines are anchored. An

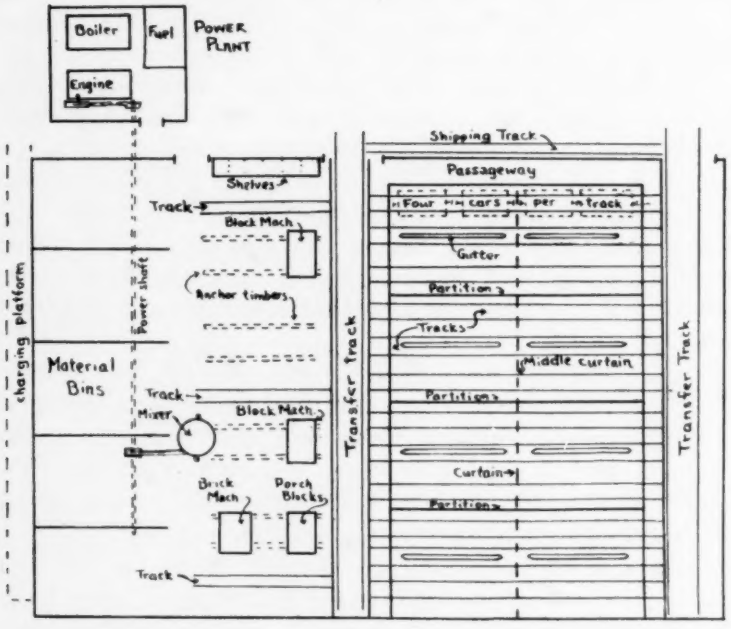


FIG. 4.

open drum mixer is located between the bins and the machines. This mixer is well adapted to this work, since the open drum may be turned toward the bins for charging and then revolved on its axis to discharge on the opposite side, next to the machines. The mixer is driven by a belt from a line shaft which extends across the building back of the machines and out into the power house, a small building immediately adjoining the main building. Pallets, interchangeable block faces and other extra parts of machines are

kept on a subdivided set of shelves at one side of this section of the building, which section might be termed the manufacturing platform.

There is a row of four curing chambers, each containing three tracks and having a capacity of twelve cars. This is a mistake in size, as previously noted, though in this particular instance the case is not so bad as it seems, as each chamber is subdivided into two end sections by a roll curtain across it at the middle. This arrangement, in connection with the subdivision of the steam piping system, enables any half chamber to be put in service independently of the rest. The chambers have cinder floors, held by concrete end sills, the theory being that the cinders will hold the moisture better and give it off slower than another form of floor.

Running along the middle of each half chamber is a shallow concrete gutter filled with water, in which there is a steam pipe perforated at one-inch intervals on the bottom. This pipe is connected at the side of the chamber with a pipe which runs up to the over-head steam main from the power house. The pipe for each half chamber is independent, so that the half section may be operated alone.

The plan of having the steam enter through a trough of water is recognized as far superior to turning steam directly into the chamber. It insures a sufficient amount of moisture at all times, and permits the use of live steam where such is more convenient. But however the steam enters, it should enter at the bottom of the chamber rather than the top, as it will diffuse through the room better in that way.

The kilns in this plant are 9 ft. high. This is a needless expense both in original construction and in steaming cost.

A plant devoted almost exclusively to the manufacture of cement tile is illustrated in Fig. 5, the large sizes being made in one building and the small sizes in another. There is no good reason why the two departments should be so widely separated except that some old buildings were utilized for the various purposes. The plan is scarcely to be commended, and yet it must be confessed that this is the plant of a very successful concern. It has its own sand pit, as will be noted in the diagram, where the sand is taken out by shovels and thrown on a belt conveyor and

thence elevated to a screen, from which it is distributed by small horse-drawn dump cars to the two departments of the plant. This plant has its own water power, which is converted into electric current at a generator house and distributed from there to the various departments. A railroad spur connects with the plant and unloads cement at the cement warehouse, as well as taking tile over the inclined track connecting with the tile yards, for such shipments as are to be made by rail. By the absolute separation of the two departments in this way the management is

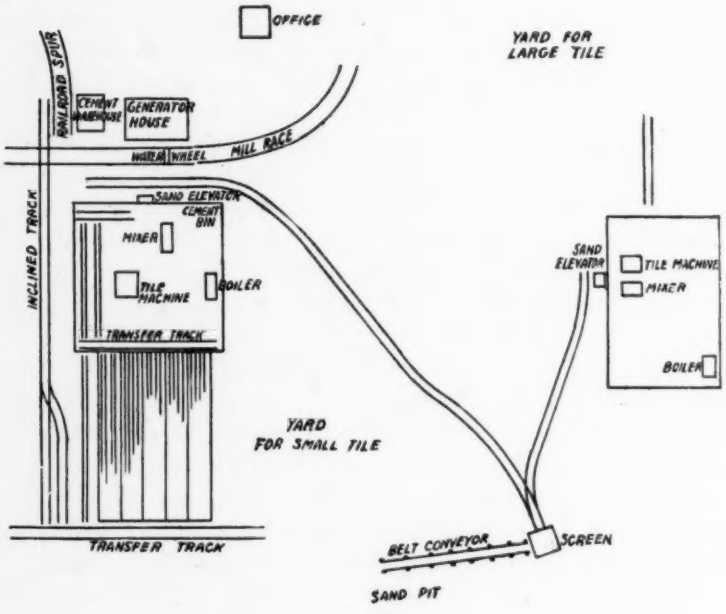


FIG. 5.

of course able to divide the cost of manufacture very accurately; at the same time the fact must be emphasized that this cost must necessarily be somewhat higher than if the plant were more closely assembled.

Fig. 6 shows a block plant installed by a large corporation for the manufacture of blocks for the construction of its own plant. The six machines installed are all alike, so that the plant consists simply of six similar units, each comprising a block machine with

a track on each side. The sand and gravel come into the plant over the track and are hoisted to an elevated bin, from which they are fed to the mixer. The mixer is also elevated and discharges into a dump car traveling on an elevated track indicated by the dotted lines. This car discharges into hoppers above each block machine, from which it is drawn into the machine as needed.

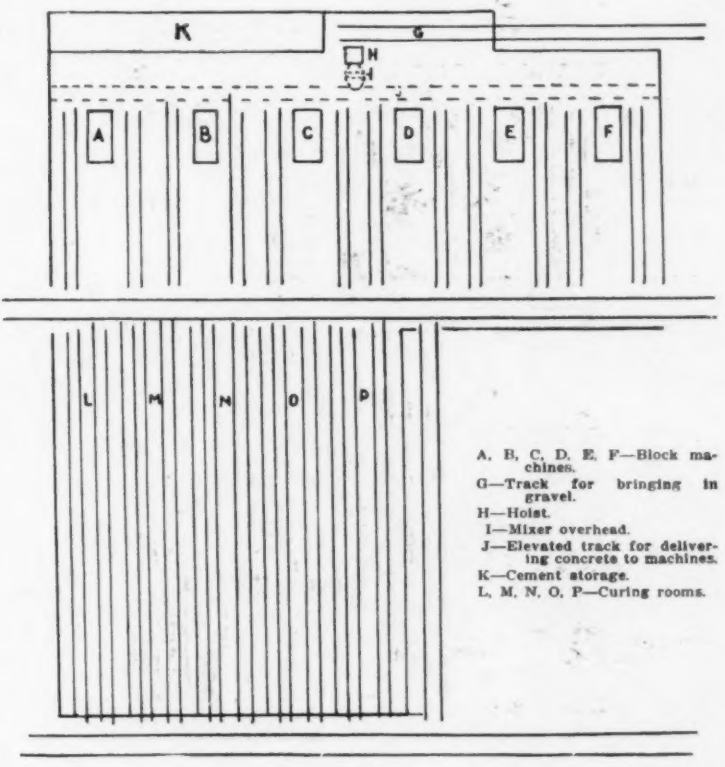


FIG. 6.

Fig. 7 shows one or two features of the arrangement of a plant for the manufacture of structural tile. It will be noted that the mixer discharges into a dump car, this car in turn delivering the concrete to three elevated troughs, each of which serves four to six tile machines. Where the car empties into the trough, the latter has the sides built up higher to prevent splashing.

In this connection, it might be advisable to call attention to one or two of the principles involved in having the mixer elevated above the working floor. This is a decided advantage in at least one respect, that is, in the delivery of the concrete, which can be mostly done by gravity. It involves, however, a larger expenditure of power for elevating the materials, and in many cases somewhat increases the labor cost, because of the fact that a man

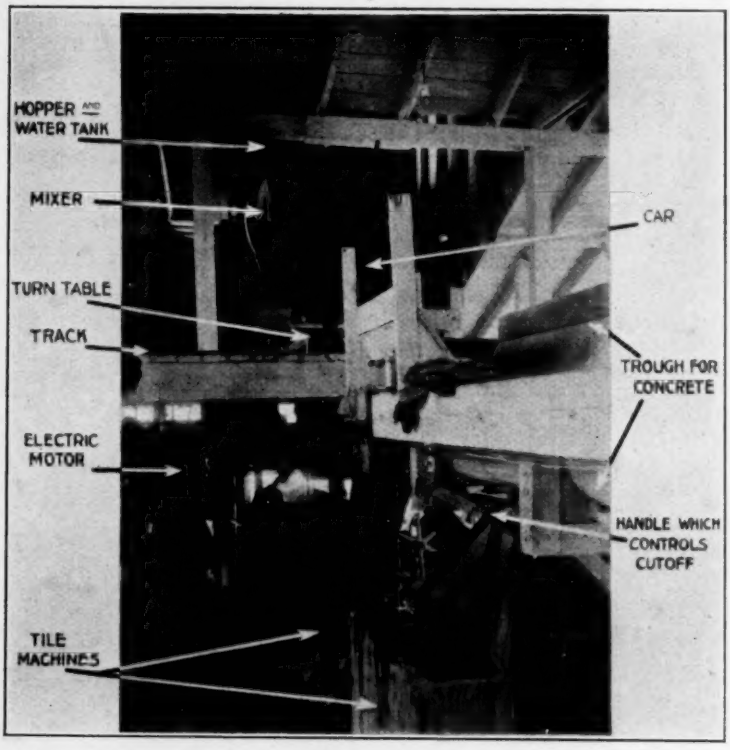


FIG. 7.

will either have to run back and forth very frequently, or else spend more of his time than is necessary in watching the mixer. While the mixer could very well be set some little distance from the working floor; there are not many instances where it could to advantage be placed on an upper floor, entirely separate from the main work-room. In a large plant, however, this might be

advisable, where more than one mixer is in operation practically all the time, requiring the continual attention of a man in any event.

The plant shown in Fig. 8 is in a two-story building and makes a variety of products. The mixing is all done on the second floor, and a chute at the side of each machine carries the concrete down ready for use. As before mentioned, this will probably be found wasteful of one man's time, in order to take care of the mixer properly and cart the concrete to the chutes. A part of the upper floor is devoted to ornamental work and one

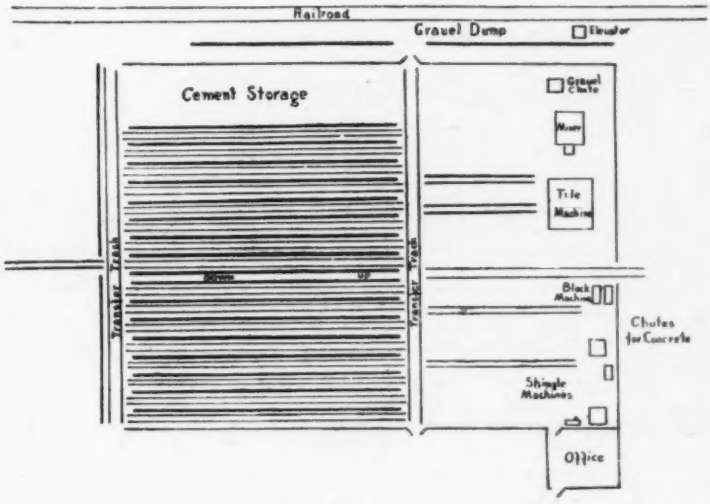


FIG. 8.

or two workmen are engaged there most of the time; but if they are suitable men for ornamental work, their time is too valuable to be spent in taking care of the mixer.

This plant, Fig. 9, is devoted to the manufacture of concrete blocks by the slush process. By this process, multiple molds are carried on cars and the facing plates are carried on the bottom of each mold. The concrete by this process is poured so wet that steam is not generally used for curing. The curing chambers shown, therefore, are not steam chambers in the ordinary sense of the word, such steam as may fill them being vaporization of

excess water from the blocks themselves. The pipes shown in dotted line run under the tracks in the curing chambers and are closed steam pipes for heating, and generally used only in seasons of low temperature. At other times the cars are simply run into the chambers and closed up to keep them from the sun and wind, so that the water may not evaporate too rapidly. In this plant the cars are run onto the transfer track under the mixer and filled by means of a chute. It will be noted in front of each curing chamber is a length of track sufficient for a car. This is for the purpose of leaving the car of blocks outside the curing chamber for an hour or two, until the initial set has taken place and part of the water has evaporated. This plant, it will be noted, has only the most elementary methods for handling material.

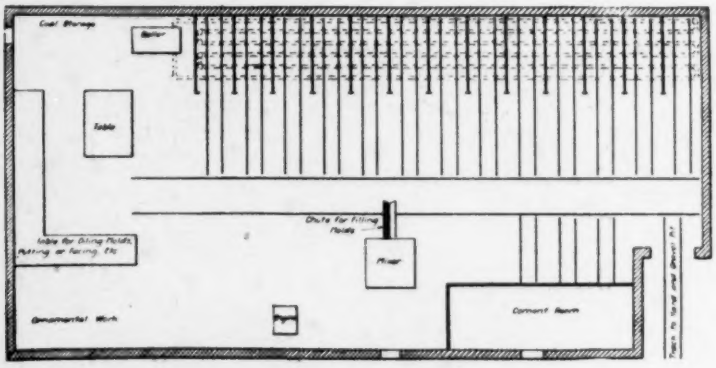


FIG. 9.

The automatic handling of materials has been given particular consideration in the construction and equipment of the plant illustrated in Fig. 10. This has been accomplished very largely by the use of screw conveyors, these being used wherever possible.

The sand is hauled in by teams and is dumped onto a grating, through which it falls to a hopper situated below the ground level. This hopper delivers to the trough of the screw conveyor, which runs under the floor of the main work-room, and from which the material is elevated by a bucket conveyor to the second floor of the building. The bucket conveyor delivers directly to the screen above a hopper, which in turn delivers to another screw

conveyor, at the discharge end of which is a measuring box by which the material is measured before going into the mixer. This box has a section of glass in the front, so that the mixer operator can see the material in the box and can allow it to fill up to a height predetermined for the particular mix he is using. In order to allow him to do this, the entire machinery which handles the gravel is at his instant control. This is made possible by electric motor drive, the motor being located in the wall some little distance away and operated by a cord close at hand.

The entire conveying machinery is operated as a unit, so that when one part is stopped, all of it stops, thus obviating the possibility of the material piling up at any one point. It is so arranged, however, that the part which takes material from the

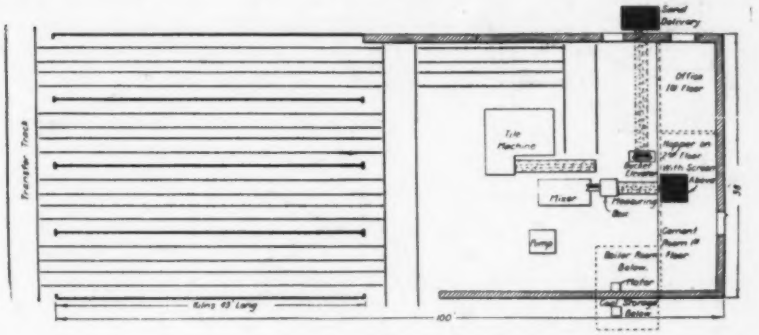


FIG. 10.

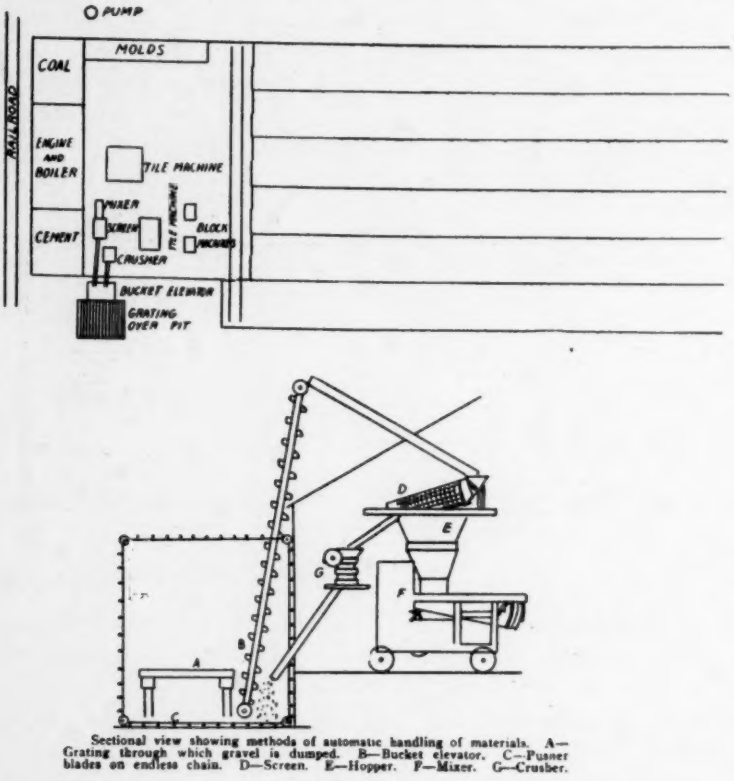
outside and elevates it to the second floor can be run independently, if desirable, without running the part which feeds the mixer. By this arrangement a considerable supply of gravel can be stored on the second floor if it is thought necessary to provide against temporary closing of the sand pit or for any other reason.

A screw conveyor is again used to convey concrete from the mixer to the tile machine. The mixer is of the tilting type and delivers directly into the trough of the conveyor, this latter in turn delivering directly to the boot of the tile machine.

Another plant of interest chiefly because of the automatic methods of handling the materials is shown in Fig. 11.

The sand is hauled in by teams and dumped onto a wooden grating built up of 2 x 6 timbers held 2 in. apart by blocks at

each end. This grating is just outside the building, and covers a pit into which the sand falls. Between the grating and the wall of the building stands a mast carrying bucket conveyor, its foot resting in the pit below and its top rising several feet above the eaves of the building, where the conveyor dumps its load into a chute. This chute leads through the roof to the mouth of a



Sectional view showing methods of automatic handling of materials. A—Grating through which gravel is dumped. B—Bucket elevator. C—Pusher blades on endless chain. D—Screen. E—Hopper. F—Mixer. G—Crusher.

FIG. 11.

revolving screen. The screen is placed directly over a sand bin, from which the material flows into the hopper of a continuous mixer, and is automatically measured out for the mix as desired.

The larger pieces of stone which are thrown out by the screen fall directly into a crusher, which is in constant motion, and are discharged from it into a chute and back to the outside pit, to be

again elevated and passed through the screen if crushed to sufficient fineness.

In order to keep the sand, in the pit below the grating, moving toward the conveyor, an endless chain is installed to run through the sand, returning on a framework overhead and carrying small paddles which keep the sand in motion. This plant makes both large and small tile and all sizes are cured by steam. This has made necessary a considerable amount of curing space and for this purpose, also, the kilns have been equipped with

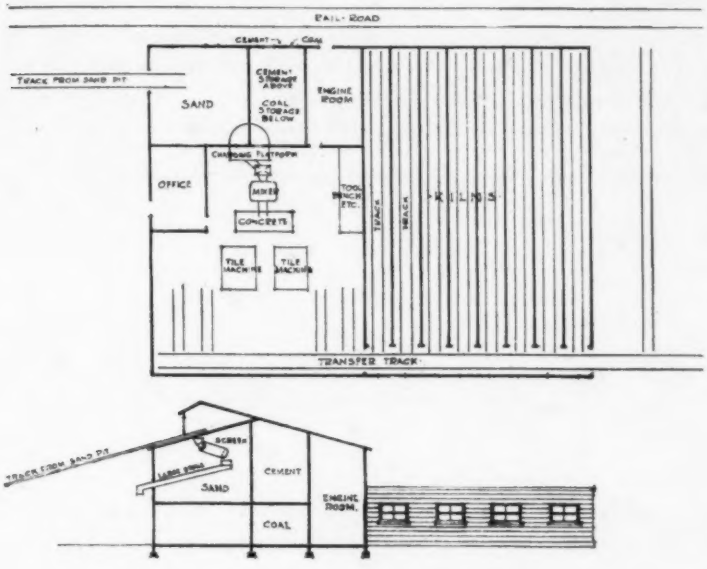


FIG. 12.

smooth concrete floors and with movable track which can be laid as needed, or taken up entirely when it is desired to set large pipe directly on the floor.

In the design of the plant illustrated in Fig. 12 some radical features have been introduced, as it was the deliberate intention to get away from some of the conventional features of such design. For instance, the kilns do not open into the main work-room, the object being to keep the steam from the curing room and entirely away from the work-room. For this reason the kilns are made parallel with the work-room instead of at right

angles with it. It is also designed to approach a square as nearly as possible, as this is usually the most inexpensive shape of building to construct. It will be noted from the sectional view that a number of windows have been suggested in the shed which covers the transfer track. These not only admit light for the workmen, but also open readily for the egress of steam. This plan was drawn with the intention of having it located close to the sand pits, with an incline track from the pit to the top of the building and dumping directly into the screens. The sand to be used is dropped directly into the bin, while the large stones are carried away in a chute and emptied outside the building. Both the sand and cement storage can be made with partial hopper bottoms and connect directly with the mixer. Provision is made for the

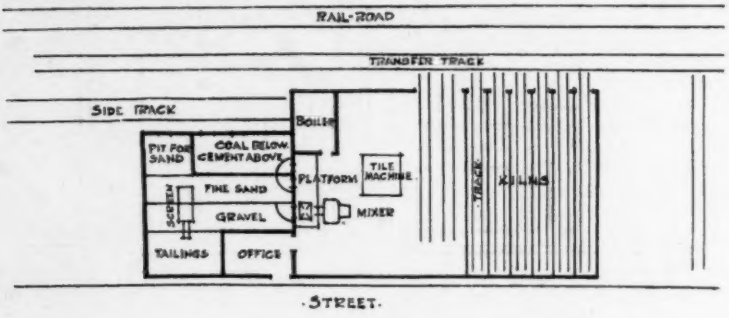


FIG. 13.

delivery of coal under the cement storage, so that both materials can be delivered from cars on the railroad track. The sand can also be delivered in the same manner should the sand pit be exhausted, or if for any reason it is desired to use a different supply.

The one-machine tile plant shown in Fig. 13 is designed primarily for location on a narrow strip of ground between a street and railroad. As in the preceding plan, the kilns do not open into the work-room. In this case, however, the cars have to go outside in order to transfer to the kilns, which would be very undesirable in some kinds of weather. The roof could be built out over the transfer track if desired.

Fig. 14 shows a sectional view of a plant which is devoted entirely to the manufacture of sewer pipe. The height of the

machine makes it necessary to feed it from the second floor, while for purposes of ready charging, the mixer is located below the level of the first floor. The materials are first proportioned in the car and then dumped into the mixer.

From the pit into which the mixer empties, a bucket conveyor carries the concrete to the second floor and dumps it into an open-end box. At each side of this box is a small pug mill, connected by a chute with one side of the double pipe machine below. The clutch which operates each pug mill is under control of the operator below, so that when a mold is in place and he is ready to begin the tamping of a pipe, the pug mill is started and immediately begins to deliver concrete down the chute. A man with a shovel is stationed on the second floor, whose duty it is to keep the pug mill filled from the open-end box.

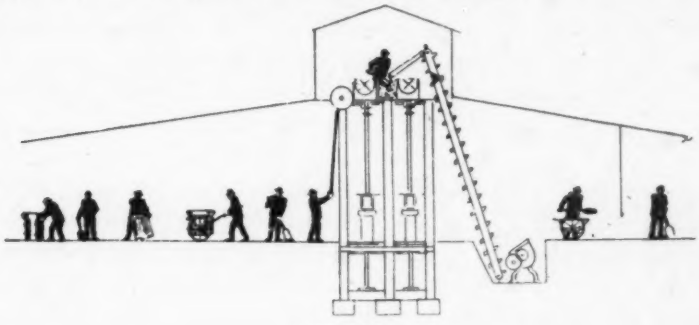


FIG. 14.

It will be seen from this that the concrete gets an extremely thorough mix—first in the mixer proper; second in the pit as it is turned over by the buckets of the conveyor; and third, in the pug mills. This puts the concrete in the best shape to be acted upon by the tampers, and makes a very dense pipe.

In conclusion it might be stated, as has been suggested before, that the ideal plant is still in the future, waiting for some man with a peculiar genius for design to apply himself to this task. The concrete product is here to stay—but not necessarily are all the manufacturers of such products in the business to stay. It is, as in other lines, a question of the survival of the fittest; and the man who has his plant designed along the lines of greatest efficiency is pretty sure to be among the survivors.

MODERN CONCRETE WORK WITHOUT FORMS.

By James E. Payne,*

The art of building concrete roofs, walls and floors without forms and with fewer shores and temporary supports than has been customary is still in its infancy.

In the past few years there have been a large number of roofs and floors built, using for forms and reinforcement some one of the many types of ribbed metals. As the cost of wood for form work is steadily increasing and the cost of steel is decreasing, rapid developments along this line may be expected.

The expanded ribbed metal sheets are made in widths of from 10½ to 28 in. and in lengths up to 12 ft. The ribs are spaced from about 3 to 7 in. apart and are from ¾ to 1½ in. in depth. The metal used is what is known to the sheet mills as soft open-hearth steel and has an elastic limit before working of 30,000 to 40,000 lb. per sq. in.

As these ribbed expanded metals not only act as reinforcement but take the place of the wood forms, in a cost comparison to determine which is cheaper, ribbed expanded metal or wood forms and wire mesh, the increasing cost of lumber is a big factor. This point coupled with a stationary or falling steel market is one reason why ribbed expanded metal slabs have grown so popular the last few years. Another reason is the time saved in the erection, and as wood forms are not needed, several floors can be placed and poured at the same time which is usually a costly method when a complete set of wood forms are used for each of several floors.

The saving in cost by using ribbed expanded metals for roof work is still greater than for floors as the complete cost of the wood form work for a roof frequently has to be charged up to the roof alone and it cannot be distributed over several floors. Roofs are built by laying these sheets on top of the purlins, lapping about 2 in. over the supports. The sheets are held down by clips fastened over every third or fourth rib and the side ribs of the sheets are fitted one over the other and punched together making the roof a

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monolithic slab. Some of these sheets are stiff enough to carry 2 in. of concrete without vibration for spans up to 4 ft., but with spans above that, one and sometimes two temporary supports are needed. The temporary supports are built by hanging a 2 x 6 or 8 across the under side of the purlins held up by wire slings wrapped around the purlins, or wedged up on the lower flange where I-beams are used. These cross pieces hold up one or two planks on edge which stiffen the metal while the concrete is being poured and while it sets.

This is essentially a short span light load system and 14 ft. is the maximum span, with the most economical span between 6 and 8 ft. For a span of 7 ft. No. 24 or No. 26 gauge sheets with 2 in. of concrete on top with $\frac{1}{2}$ in. of cement plaster below will carry any ordinary roof load. A roof of this kind averages between 15 and 20 cts. per sq. ft. exclusive of structural steel and waterproofing. As floors are usually thicker than roofs, the cost increases proportionately.

The concrete is specified to be made of a 1 : 2 : 4 mix with stone or gravel to pass a $\frac{1}{2}$-in. ring with the dust screened out, but is usually made according to the accepted standards for concrete work in that locality. When the concrete has set enough to carry itself the wires are cut and the supports dropped leaving the under surface of the slab ready for plaster. These temporary supports seldom cost more than 1 ct. per sq. ft., including, lumber-placing and removing.

The plaster used is a cement plaster mixed about 1 to $2\frac{1}{2}$ with a small amount of hair and only enough lime (not more than 5 per cent. of the volume of cement and sand) to make the cement work easily under the trowel. For factory buildings this finish is sometimes floated, but for offices, hospitals, hotels, etc., the white coat is frequently added. The trouble of making the plaster stick to a reinforced concrete slab usually experienced, cannot occur with ribbed expanded metals, because the concrete projects through the mesh to some extent, and that with the mesh formation makes an excellent key for the plaster.

How do you keep the concrete from running through the reinforcement is a natural question considering the care taken to make wood forms watertight. The concrete used with open mesh ribbed expanded metal is dryer than that used with wood forms. The mix is sometimes known as a quaking mixture and

when dumped from a barrow tends to stay where dumped and is spread with a shovel or hoe. The material that drips through the mesh is principally water and contains very little cement. In a factory building erected this summer in Youngstown, 290 ft. long, the contractors sprinkled sand on the finished cement floor below the roof, and as the roof was poured a man below shoveled up the sand and carted it out. The drip from the roof was not enough to set up the sand. It seems as if the mesh curls the mortar around itself something like the way metal lath does with plaster. In a test conducted recently to determine the amount of loss through dripping it was shown that the loss amounts to from 36 to 80 oz. from an area of $3\frac{1}{2}$ sq. ft. In percentage this ranges from 2.6 to 4.3 per cent loss, depending on the style of mesh between the ribs.

One of the most important advantages of using these steel sheets for combined forms and reinforcing, is that there is no waiting for the removal of forms from one floor to start the construction of another, and the delay in pouring concrete, occasioned by the time taken to place and wire the reinforcement, is saved entirely as the concrete is poured as soon as the expanded metal forms are in place. In the Youngstown City Hall having 6 floors, an attic and a roof, all the sheets were placed and floors poured in 30 days, an average of less than 4 days to a floor. This is an important matter on penalty jobs, and also in the fall, in order to get the roof on before the snow flies. In practically all of the reported failures in reinforced concrete, the too early removal of forms is a contributing cause, if not the sole reason, for the collapse. This great danger is eliminated by using materials that combine the two functions of forms and reinforcement.

Another use for ribbed expanded metal is in the construction of inclined slabs. They have been built having an angle of as much as 60 deg. from the horizontal. Formerly to build a slab like that, it was necessary to have forms on both sides. By using a dry mix and running the ribs in a horizontal direction each rib acts as a baffle to prevent the concrete from sliding down the slope. The roof over the new hotel of the Union Stock Yards and Transit Company in Chicago is an example of this kind.

It sometimes happens that ribbed expanded metals are used for forms only—and no attention is paid to the reinforcing value. In the roof over the Washington Theatre in Detroit, the cost of

erecting and removing wood forms would have been very high, as the roof was 50 or 60 ft. above the pit. Ribbed expanded metal was used on top of the purlins and rod reinforcement placed above the metal. The concrete was poured and the under surface left unplastered. To plaster the underside of that roof, scaffolds would have been hung from the roof beams or built up from below at the time the roof was poured or shortly afterward. The cost of the additional reinforcement of rods was much less than the cost of the other alternative which necessitated the scaffold remaining in place several weeks or possibly months waiting for the building to reach a condition which would permit the use of the scaffold by the ornamental plasterer.

For large areas the objection has sometimes been made that these materials could not be used in connection with the gravity system of distributing concrete, because of the large amount of water needed to properly work the gravity system, and consequently the large loss through the expanded section of the mesh. For example, a roof 100 ft. square needs theoretically 61.7 yards of concrete for a roof 2 in. thick. A 5 per cent loss through the mesh, 3 yd. of concrete at $6.50 per yd., amounts to $19.50. Probably a fair average of saving secured by using the gravity system over wheelbarrows and runboards is 75 cts. per yd. or a saving in this roof of $46.00. These figures show the falsity of the idea entertained by some engineers that the gravity system must be discarded if ribbed expanded metals are used as forms.

The above remarks have referred entirely to flat sheets but the use of curved sheets is almost as large. It frequently happens that a curved or arched floor is wanted, but heretofore it has been almost prohibitive because of the excessive cost of labor in making arched or semi-circular forms. By arching these sheets and resting the ends on the lower flanges of the steel beam or on the wood sides, or bottom of the beam box, the sheets take the place of curved forms and also stiffen the concrete in the arch. These arched floors are quite popular in breweries and also other warehouses where heavy loads are liable to be applied. They are also well fitted for highway bridges.

In the Buhl Country Club at Sharon, Pa., the beams were 8 ft. 8 in. apart and 24 gauge curved sheets were placed resting on the lower flanges of the I-beams. The rise of the arch was 16 in. and the concrete was poured with no temporary supports. This system

is well fitted for cases where the under surface is difficult of access, as the concrete arch carries the load, the steel sheets acting only as permanent forms, and also for cases where heavy loads will be carried.

Cement non-bearing partitions were erected more cheaply by plastering cement mortar on ribbed expanded metal than by a double wall of wood forms with poured concrete between. The time lost in waiting for the removal of the wood forms in one place and their erection in another is saved completely as the sheets are ready to be plastered as soon as erected. Almost all other fireproof partitions are at least 4 in. thick when finished, some as much as 6 in. Solid plastered partitions are only 2 in. thick, which means that every lineal yard of partition erected adds a square foot to the floor space.

When made of cement, these 2 in. solid partitions are as fireproof, if not more so, than any other partition in common use. In Cleveland at the laboratories of the Associated Metal Lath Manufacturers, a 2-in. solid partition 7 ft. wide by 9 ft. high was built plastered on ribbed expanded metal. On Novemeber 20, 1912, it was tested, the fire burning for two hours with a temperature averaging 1849 deg. after the first half hour.

While the partition showed some deflection toward the fire, and a number of cracks on the side away from the fire, no smoke came through and when the panel was swung open, there were but two small cracks on the side exposed to the fire. Water at Cleveland City pressure was then turned on for $2\frac{1}{2}$ minutes, and while sections of plaster were washed off on the fire side, the water did not go through the partition, and it was evident that a new coat of plaster was the only thing needed to render it as fireproof as ever.

Twenty-eight gauge sheets and 2 in. of solid cement plaster has proven entirely satisfactory for interior partitions up to 12 ft. high, and 26 gauge sheets and $2\frac{1}{2}$ in. of concrete for partitions up to 17 ft. The sheets are fastened to the floor and ceiling with a runner bar or angle which has been previously placed and lined up, the sheets being wired or clamped to the angle. Two x 4 in. temporary stiffeners are wired to one side of the sheets about 4 ft. apart, and are then braced back to the floor. When the opposite side is scratched in the supports are removed and that side of the partition is plastered. The baseboard, chair rail and picture mold are nailed into small blocks which are wired to the steel between the

ribs at intervals and used to hold the grounds. Some of these metals are made in such a way so that a $\frac{3}{4}$-in. electric conduit fits in between the ribs and is imbedded in the plaster. The metal is cut away as much as necessary to allow the outlet box to be placed.

The average cost is between $1.25 and $1.75 per sq. yd. complete in the populous sections of the country and this is usually less than the cost of burned clay tile partitions.

Probably the least expensive silo of all the fireproof silos now on the market can be constructed of ribbed expanded metals plastered with cement. The sheets are curved at the factory and temporarily held in place by wood uprights on one side which are removed as soon as the scratch coat hardens. By punching the ends of the ribs together so that the joint is as strong as the rib, and lapping the ends enough to develop the full strength of the mesh, the steel sheets have sufficient sectional area to resist the bursting pressure developed by the ensilage.

This same method is used for small water tanks, but for larger tanks, it has been found that a better result is secured by building two walls of curved ribbed expanded metal, and pouring between with concrete. In this way, enough additional reinforcing can be introduced to take up the stresses over and above those taken care of by the ribbed metal.

It is generally thought that the published load tables showing the carrying capacity of flat slabs of this character, are based on past experience and judgment only, but this is not entirely so. They can be computed with as much mathematical certainty as any other type of reinforced concrete.

In determining the tensile strength of a foot of ribbed expanded metal, the area of all the metal in the ribs, and also such metal between the ribs that runs continuously from one support to another, is to be considered as resisting tension. As most of the metal is at the lower edge of the sheet, the center of gravity of the tensile reinforcement instead of being at the center of the steel as is the case when bars are used, is located about two-thirds of the distance from the top of the rib.

The resisting moment per foot of width of the slab, is the tensile strength of the steel per foot of width, multiplied by the distance between the center of gravity of the steel, and the center of gravity of the compressive forces in the concrete.

Floor and roof slabs are designed according to accepted

engineering practice, except that they are usually considered to be simply supported at the ends and are not reinforced against negative bending stresses over the supports. A test was made October 26, 1912, under the control of the Cleveland Building Department, on two spans of 6 ft. each. This test was expected to prove that the accepted methods of design could be applied to slabs reinforced with sheet metal of certain patterns. Slab No. 1 was 2 in. thick and designed to carry a working load of 126 lb. When loaded to 2½ times that, or 329 lb. per sq. ft., the deflection was .16 in., which increased to .20 in. 24 hours later. Slab No. 2 was 2½ in. thick and designed for a working load of 156 lb. Its deflection at 2½ times that was .16 in. and .20 in. 24 hours later. These tests substantiated the claims made by the manufacturer with a large excess of strength to spare.

In a sheet of No. 24 gauge expanded ribbed metal, the surface of steel in contact with the concrete is about five times the surface of a round bar of equal cross-sectional area. In ordinary bar and slab construction where the bond stress on the bars seldom equals 125 lb. per sq. in. the bond stress in the ribbed metal would be only 25 lb. per sq. in. of surface. This low bond stress is taken advantage of by shipping painted sheets instead of plain, which prevents incidental rust during transportation. The expanded section of the sheet is also gripped by the concrete, so practically all sides of the metal in the mesh is covered, and this will reduce the bond stress still lower than 25 lb. per sq. in.

Numerous attempts have been made to increase the efficiency of those different metals by the spacing and height of the ribs and also changing the type of mesh between the ribs.

It will readily be seen that to increase the depth of the ribs, is to stiffen the sheet so the temporary supports can be spaced further apart, but when the ribs are made deeper, the center of gravity of the steel is raised, requiring an increased thickness of the concrete slab, and the saving in the cost of temporary supports is more than offset by the additional concrete required.

In conclusion, it appears that the high quality of this type of construction, with its low cost, coupled with the fact that the features of design are so simple and sure, would warrant the careful consideration of all engineers and architects, and as a possible future composition for inflammable roofs deserves their active support.

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